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Current ACI Standards

(As of December, 1949)

Listed below are ACI standards in effect at the time the closing portions of *Proceedings* V. 45 are published in the ACI JOURNAL, Part 2 December 1949:

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)

ACI JOURNAL, Sept. 1949, *Proc.* V. 46, pp. 1-16. In separate covers $0.50; to ACI Members $0.40.

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Not adaptable to Journal presentation, this is a separate publication of large format bound to lie flat. It presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. 55 pages, $3.00; to ACI Members $1.75.

Recommended Practice for Winter Concreting Methods (ACI 604-48)

ACI JOURNAL, Sept. 1948, *Proc.* V. 45, pp. 1-20. In separate covers $0.50; to ACI Members $0.40.

Building Code Requirements for Reinforced Concrete (ACI 318-47)

ACI JOURNAL, Sept. 1947, *Proc.* V. 44, pp. 1-64. In separate covers $0.50; to ACI Members $0.40.

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)

ACI JOURNAL, Oct. 1946, *Proc.* V. 43, pp. 133-148. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)

ACI JOURNAL, Oct. 1946, *Proc.* V. 43, pp. 149-164. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 651-672. In separate covers $0.50; to ACI Members $0.40. Also available in a Spanish edition: "Normas Practicas Recomendadas Por el ACI Para Proyectar Mezclas de Hormigon (ACI 613-44)." In separate covers $0.50.

Specification for Cast Stone (ACI 704-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 701-704. In separate prints $0.35.

Specifications for Concrete Pavements and Bases (ACI 617-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 673-700. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 625-650. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 621-624. In separate prints $0.35.

The AMERICAN CONCRETE INSTITUTE

is a nonprofit, nonpartisan organization of engineers, architects, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete, dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

Since 1905 that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practice in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

AMERICAN CONCRETE INSTITUTE
New Center Building, Detroit 2, Michigan

Chairman Spencer and the members of his committee give us a new standard—ACI 604-48 with its "recommended practices for cold weather concreting for buildings, pavements and other thin sections" and also for "mass concrete." Valuable work of Committee 604 is in the appendix, which presents "whys and wherefores" in data supporting the objectives of such winter operations, and a copious bibliography.

ACI Standard

Recommended Practice for Winter Concreting Methods (ACI 604-48)*

Reported by ACI Committee 604

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SYNOPSIS

This ACI Standard establishes standard methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RECOMMENDED PRACTICE FOR COLD-WEATHER CONCRETING FOR BUILDINGS, PAVEMENTS AND OTHER THIN SECTIONS

Temperature at which protection is required

Planning for cold weather should be done well in advance of its occurrence. When experience or weather records indicate that low temperatures are probable, a plan for protecting all concrete at early ages should

*Adopted as a standard of the American Concrete Institute at its 44th annual convention, February 25, 1948 as reported by Committee 604; ratified by letter ballot June 29, 1948. Title No. 45-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 1, *Proceedings* V. 45. Title No. 45-1 supersedes 44-13 published in December 1947. Separate prints are available at 50 cents each. **Discussion** of this paper (copies in **triplicate**) should reach the Institute not later than January 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Proxy for Charles E. Wuerpel during the latter's absence from the United States.

be decided upon and the necessary special equipment and materials provided at the site of the work before the low temperatures occur.

Whenever it is anticipated that the air temperature at the point of placement is likely to fall below 40 F during the 24-hour period after placing concrete or below 30 F during the succeeding 6 days, it is recommended that protective measures be taken as outlined in the following paragraphs.

Heating of materials

For air temperatures not lower than 30 F, the mixing water should be heated to bring the temperature of the concrete at the mixer to between 50 F and 80 F or, alternatively, for small jobs only, (less than 20 cu yd of concrete) the concrete may be heated by means of a torch inside the mixer.

For air temperatures of from 30 F to 0 F both the water and the fine aggregate should be heated to bring the temperature of the concrete at the mixer to between 50 F and 90 F. Only coarse aggregate that is free from frozen lumps should be used.

For air temperatures below 0 F the water, the fine aggregate and the coarse aggregate should all be heated to eliminate frost and to bring the temperature of the concrete at the mixer to between 50 F and 90 F.

Mixing water should be heated under such control and in sufficient quantity so that appreciable temperature fluctuations from batch to batch are avoided. To avoid flash set where either aggregate or water is heated to a temperature in excess of 100F, the loading of the mixer should be in such sequence that the cement does not come in contact with such hot materials.

Aggregates should be heated in a manner such that frozen lumps, ice and snow are eliminated and overheating or excessive drying is avoided. At no point should the aggregate temperature exceed 212 F and the average temperature of an individual batch of aggregate should not exceed 150 F. The use of steam coils is recommended for heating aggregate, but for small isolated jobs, aggregates may be thawed by heating them carefully over culvert pipes in which fires are maintained.

When aggregates in stock piles, cars or trucks are thawed or heated by means of steam coils, the exposed surface should be covered with tarpaulins as much as practicable to maintain a uniform distribution of heat and prevent the formation of frozen crusts.

Accelerators and anti-freeze compounds

The use of small amounts of additional cement or accelerators such as calcium chloride to accelerate the hardening of the concrete at low temperatures (above 32 F) may be permitted but the use of large amounts

of additional cement or more than 2 percent of calcium chloride to accelerate the hardening of the concrete or to secure higher than normal strength at early ages should be permitted only after careful consideration of the possibility of future injurious effects.

The use of salts, chemicals or other foreign materials in the mix to lower the freezing point of the concretes should not be permitted.

Curing temperature

Newly placed concrete made with normal cement should be kept at a temperature of not less than 50 F for 7 days or not less than 70 F for 3 days. It is good practice to maintain a temperature of at least 40 F for the next 4 days. At the end of the curing period artificial heating should be discontinued and housings removed in such a manner that the fall in temperature at any point in the concrete will not exceed 40 F in 24 hours.

Concrete made with high early strength cement should be kept at not less than 70 F for 2 days or not less than 50 F for 3 days.

The surface temperature of the hardened concrete should not be permitted to exceed 100 F at any time during the curing period. The methods of protection and curing should be such that the loss of moisture from the concrete is not excessive during the curing period.

Temperature records

A permanent record should be kept showing the date, hour, outside air temperature and temperatures at several points within the enclosure and at the concrete surface to show the highest and lowest temperatures of the concrete surface. The use of thermometers embedded in the concrete surface is suggested. Thermometer readings should be taken at the start of the work in the morning and again in the late afternoon. The data so obtained should be recorded in such a manner that the location of each reading and any conditions which might have an effect on the temperature will be shown. A copy of the temperature record should be preserved for future reference.

Preparation before concreting

Before concrete is placed in any form or around any reinforcement or on any surface, all ice, snow and frost should be completely removed and the temperature of all surfaces in contact with the concrete raised above the freezing point. No concrete should be placed on a frozen subgrade or on one that contains frozen materials.

Protection

Arrangements for covering or housing the newly placed concrete should be made in advance of placement and should be adequate to maintain in all parts of the concrete the temperature and moisture con-

ditions recommended in the paragraph on curing temperatures. When air temperatures do not fall below 30 F, artificial heat will not be required for pavement, sidewalks or other flat slabs on the ground if the surface is well protected. Such protection might consist of insulating mats or the covering of the surface with canvas on top of at least 8 in. of loose hay or straw placed as soon as the concrete has hardened sufficiently to permit the application without damage. For lower temperatures, sufficient well distributed artificial heat should be provided inside the coverings or enclosures around the concrete to maintain a temperature of the concrete surface of 40 F or above at the coldest point without exceeding 100 F at the hottest point.

Housings may be made of wood, canvas, Sheetrock, Celotex, Sisalkraft, tarred paper, plywood or other suitable materials, as long as they are reasonably tight and safe from wind and snow loading. Housings should be built with fire resistant construction as far as practicable. Adequate fire fighting apparatus should be furnished in easily accessible locations wherever enclosures are constructed. Attendants should be maintained on the job at all times to provide continuous operation of heating units.

In placing floor slabs, tarpaulins or other readily moved coverings supported on horses or framework should follow closely the placing of the concrete so that only a few feet of finished slab are exposed to the outside air at any one time. Such tarpaulins should be arranged so that heated air can be circulated freely on both top and bottom of the slab. Layers of Celotex placed directly on the concrete have also been found to be effective in preventing freezing.

Housings and enclosures should be left in place for the entire curing period specified except that sections may be temporarily removed as required to permit placing additional forms or concrete, provided they are replaced as soon as the form or concrete is in its final position.

Removal of forms

Supports for concrete members should remain undisturbed until the concrete has attained sufficient strength to sustain its own weight in addition to any temporary or permanent load that may be placed upon it during the building of the structure. Beam sides, column forms or forms for walls may be removed as soon as the concrete has hardened enough to support its own weight, provided that such action does not endanger any part of the structure.

Prior to the proposed removal of any form or centering, the approval of the engineer responsible for the design of the structure should be secured. Before giving his approval the engineer should give due consideration to the date on which the conrcete in this particular section

was placed, the record of temperature conditions prevailing at that section during the interval since placement, the probable strength of the concrete and any pertinent facts concerning the proposed future loading of the section in question.

If, in the opinion of the engineer, based on the records and the condition of the concrete, the removal of the forms is likely to endanger the whole or any part of the structure, the forms should remain in place for such an additional period as may be necessary to insure safety. Proper re-shoring should be installed for the support of all members as required. These re-shorings should remain in place until all danger of overloading the members supported is past.

RECOMMENDED PRACTICE FOR COLD-WEATHER CONCRETING FOR MASS CONCRETE

Temperature at which protection is required

Planning for cold weather should be done well in advance of its occurrence. When experience or weather records indicate that low temperatures are probable, a plan for protecting all newly-placed concrete should be decided upon and the necessary special equipment and materials provided at the site of the work before the low temperatures occur.

Whenever it is anticipated that the air temperature at the point of placement is likely to fall below 35 F in the 24-hour period after placing concrete, it is recommended that protective measures be taken as outlined in the following paragraphs.

Heating materials

For air temperatures not lower than 30 F the mixing water should be heated to bring the temperature of the concrete at the mixer to between 40 F and 60 F.

For air temperatures below 30 F both the water and the fine aggregate should be heated to bring the temperature of the concrete at the mixer to between 40 F and 70 F. The coarse aggregate should be free from frozen lumps or ice.

Mixing water should be heated under such control and in sufficient quantity so that rapid temperature fluctuations of the water are avoided. Water temperatures should not be permitted to exceed 140 F.

Aggregates should be heated in a manner such that frozen lumps, ice and snow are eliminated and overheating or excessive drying is avoided. At no point should the aggregate temperature exceed 212 F. The use of steam coils is recommended for heating aggregate.

When aggregates in stock piles, cars or trucks are thawed or heated by means of steam coils, the exposed surface should be covered with

tarpaulins as much as practicable to maintain a uniform distribution of heat and prevent the formation of frozen crusts.

Curing temperature

Newly-placed concrete should be kept at a temperature of not less than 40 F for 14 days. The temperature of the concrete surface should not be permitted to exceed 100 F at any time during the curing period. The methods of protection and curing should be such that the loss of moisture from the concrete is not excessive during the curing period.

Temperature records

A permanent record should be kept showing the date, hour, outside temperature, and temperatures at several points within the enclosure and at the concrete surface to show the highest and lowest temperatures of the surface. The use of thermometers embedded in the concrete is suggested. Thermometer readings should be taken at the start of the work in the morning, in the late afternoon and again at midnight. The data so obtained should be recorded in such a manner that the location of each reading and any condition which might have an effect on the temperature will be shown. A copy of the temperature record should be preserved for future reference.

Anti-freeze compounds

The use of salts, chemicals or other foreign materials in the mix to lower the freezing point of the concrete should not be permitted in mass concrete.

Preparation before concreting

Before concrete is placed in any form or around any reinforcement or on any surface, all ice, snow and frost should be completely removed and the temperature of all surfaces in contact with the concrete raised above the freezing point. No concrete should be placed on a frozen subgrade or on one that contains frozen materials.

Protection

Arrangements for covering or housing the newly placed concrete should be made in advance of placement and should be adequate to maintain in all parts of the concrete the temperature and moisture conditions specified in the paragraph on curing. When air temperatures do not fall below 30 F, artificial heat will not be required if the concrete surface is covered with forms or with canvas arranged so that an air space is maintained between the canvas and the concrete. For lower temperatures, sufficient well distributed artificial heat should be provided inside the coverings or enclosures around the concrete to maintain a temperature of 40 F or above at the coldest point without exceeding 80 F at the hottest point.

Housings may be of wood, Sheetrock, Celotex, canvas, tarred paper, plywood or other suitable materials, as long as they are reasonably tight and safe from wind and snow loading. Housings and enclosures should be built with fire resistant construction as far as practicable. Adequate fire fighting apparatus should be furnished in easily accessible locations wherever enclosures are constructed. Attendants should be maintained on the job at all times to provide continuous operation of heating units.

Housings and enclosures should be left in place for the entire curing period specified except that sections may be temporarily removed as required to permit placing additional forms or concrete, provided they are replaced as soon as the form or concrete is in its final position.

At the end of the curing period, artificial heating should be discontinued and housing removed in such a manner that the fall in temperature at any point in the concrete will not exceed 20 F in 24 hours.

Removal of forms

Forms for unreinforced mass concrete should remain undisturbed until the concrete has attained sufficient strength to sustain its own weight in addition to any temporary or permanent load that may be placed upon it during the building of the structure.

The time of removal of forms or supports for reinforced concrete structural members should be subject to the approval of the engineer responsible for the design of the structure.

APPENDIX

Winter concreting objectives

Special concreting methods are necessary during cold weather because the strength and durability of concrete are adversely affected when it is exposed to freezing temperatures during the period of placing and curing. The methods must be adequate to:

1. Prevent damage to concrete from freezing and thawing at an early age;
2. Allow the concrete to develop early strength, permitting the removal of forms for prompt re-use and the loading of the structure when desired;
3. Maintain proper curing conditions;
4. Limit excessive or rapid temperature changes before the strength of the concrete has developed sufficiently to resist temperature stresses;
5. Obtain the degree of economy consistent with the design or intended use of the concrete.

1. *Prevention of damage at early ages*—The exact limits of the critical period when concrete may be seriously damaged by one or two cycles of freezing are uncertain and are doubtless affected by many factors in the composition and manufacture of the concrete. Nevertheless, the major factor affecting the limit of the critical period is the rate of strength gain of the concrete. Experience and meager laboratory tests indicate that if the concrete has attained a compressive strength of over 500 psi it will not be seriously damaged by one or two freezing cycles. On account of the impracticability

of making truly representative compressive tests of all portions of a structure it is necessary to set limitations on placing and curing temperatures that will insure the designed strength in even the weakest part of the concrete in each class of concrete specified.

The most reliable means of protecting concrete from frost damage at early ages is to build a temporary housing to enclose the whole volume to be occupied by the concrete structure and to maintain proper curing temperatures in the enclosure by means of artificial heat. This has been done for certain important structures of limited size which had to be built in extremely cold weather. On account of the cost, such a method is not economical in most cases. Practical alternatives are the construction of canvas, plywood, Sheetrock, Celotex, Sisalkraft and other weatherproof enclosures around the forms in which the concrete is to be placed, and the maintenance, for the full curing period, of proper curing temperatures within these enclosures by means of steam unit heaters, steam jets, steam pipes, salamanders or other heaters.

Under occasional specially favorable conditions, concrete may also be protected from frost damage at early ages by placing the concrete at a temperature high enough so that the initial heat plus the heat generated by the hydration of the cement is sufficient to balance the loss of heat to the outside without lowering the surface temperature to the freezing point. It is obvious that the method is limited to air temperatures within a few degrees of freezing. It is also clear that if no protection is provided newly placed concrete is liable to be damaged during any sudden drop in temperature. As is pointed out in a following section, the use of hot concrete is apt to lead to excessive cracking when the concrete cools. Furthermore, the use of concrete which is hot as it comes from the mixer results in greatly decreased workability and a tendency toward flash setting.

2. *Production of enough strength to permit removal of forms or loading of the structures*—Winter concreting methods should be arranged to produce the proper concrete strength; first, to permit economical re-use of forms; second, to permit construction of the upper portions of the structure; and third, to permit loading the structure when desired. The strength required for each case will depend on the type of structure. The value of early form removal and early use of the structure must be balanced against the cost of additional heating and protection in cold weather. The effect of curing temperature on concrete strength can be found conveniently in Fig. 1 to 5 which are taken from the JOURNAL of the American Concrete Institute for November-December 1934. In considering the data in the charts it should be kept in mind that the results shown were based on tests of 3 x 6-in. concrete cylinders. Thus, the volume of concrete was relatively small and the time element required to bring the concrete to the temperature of the surrounding air much shorter than would occur in the field even for thin sections.

The use of high early strength cement should be considered from the standpoint of overall economy. When high early strength cement is used, the period during which temperature protection and moist curing must be maintained may be reduced as indicated by the charts referred to in the preceding paragraph.

The use of calcium chloride or other chemicals as accelerators for the hardening of the concrete may likewise be considered from the standpoint of economy under the limitations mentioned in the preceding paragraph. However, the use of calcium chloride in excess of 2 percent of the weight of the cement should be discouraged. Furthermore, there is some evidence that calcium chloride intensifies the destructive reaction between the alkalies in portland cement and certain susceptible aggregates. A good discussion of the use of calcium chloride as an accelerator is found in Section 1 of the report by ACI Committee 212 on "Admixtures for Concrete". The report appeared in the ACI JOURNAL, November 1944, *Proc.* V. 41, p. 73.

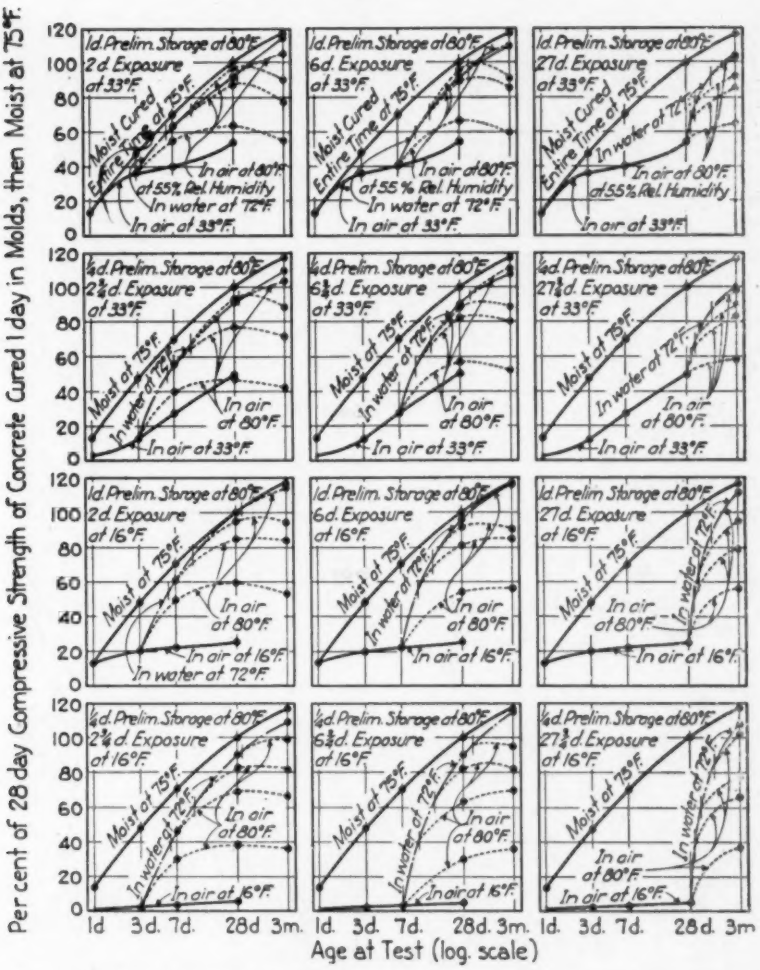

Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then Moist at 75°F.
1d. Prelim. Storage at 80°F. 2 d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 2d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 16°F.
Moist Cured Entire Time at 75°F.
Moist at 75°F.
In water at 72°F.
In air at 80°F. at 55% Rel. Humidity
In air at 80°F.
In air at 33°F.
In air at 16°F.
0
20
40
60
80
100
120
1d 3d. 7d. 28d 3m.
Age at Test (log. scale)

Fig. 1—Relative strength of concrete as influenced by warming both in water and in air after exposure to temperatures of 16 and 33 F—normal cement

From Timms, A. G. and Withey, N. H., "Further Studies of Temperature Effects on Compressive Strength of Concrete", ACI JOURNAL, Nov.-Dec. 1934, *Proc.* V. 31, p. 165.

The use of calcium chloride, common salt or other soluble materials for the purpose of preventing freezing solely by lowering the freezing point of the mixing water should not be permitted. Calcium chloride should not be used as a substitute for proper curing and frost protection.

3. *Maintenance of proper curing conditions*—An adequate supply of moisture should be maintained on all exposed concrete surfaces during the curing period to insure the development of the proper strength and surface hardness.

4. *Limitation of rapid temperature change during and immediately after the curing period*—It is a well-known fact that excessive temperature changes or excessive differences in temperature within a body of concrete promote cracking and have a harmful

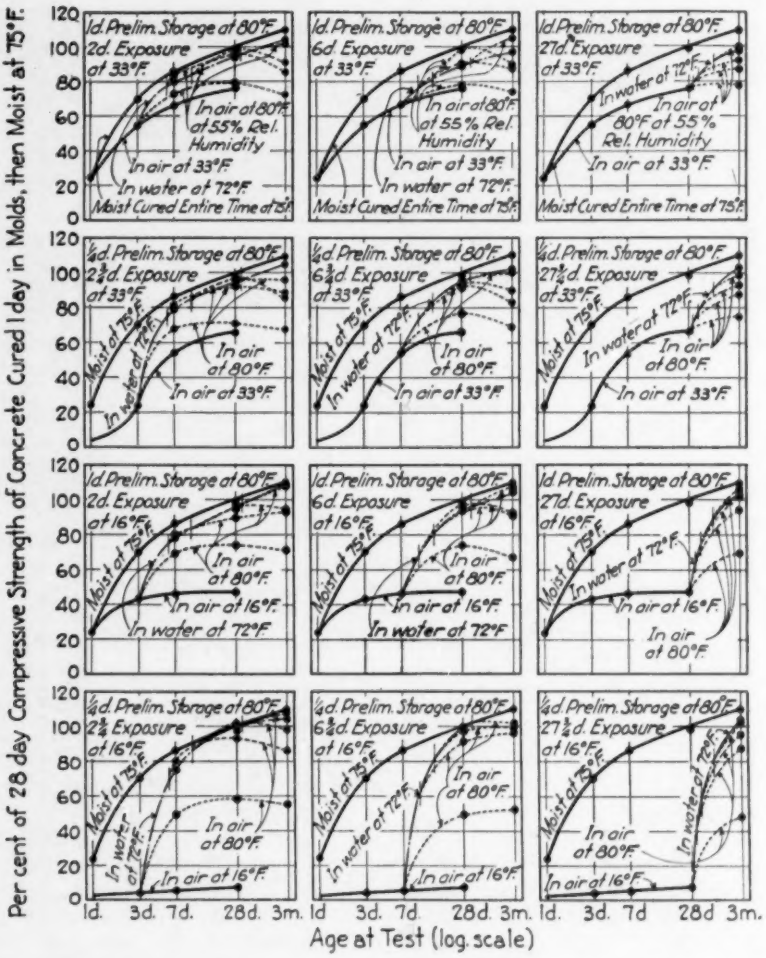
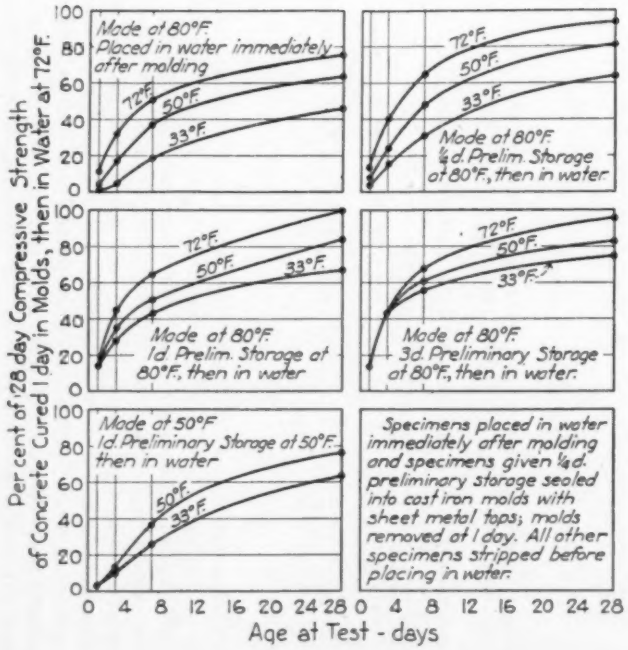

Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then Moist at 75°F.
Age at Test (log. scale)
1d. 3d. 7d. 28d. 3m.
1d. Prelim. Storage at 80°F. 2d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 2d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 2¾ Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 16°F.
In air at 80°F. at 55% Rel. Humidity
In air at 33°F.
In water at 72°F.
Moist Cured Entire Time at 75°F.
Moist at 75°F.
In air at 80°F.
In air at 16°F.

Fig. 2—Relative strength of concrete as influenced by warming both in water and in air after exposure to temperatures of 16 and 33 F—high early strength cement

From Timms, A. G. and Withey, N. H., "Further Studies of Temperature Effects on Compressive Strength of Concrete", ACI JOURNAL, Nov.-Dec. 1934, *Proc.* V. 31, p. 165.

effect on strength and durability of a structure. These effects become particularly important at the time of form removal and at the time of discontinuance of protection against low temperatures. At such times, the surface of the concrete may be rapidly chilled and cracking may result. The seriousness of the cracking due to sudden chilling at the surface is greatest in mass concrete where the heat of hydration of the cement may raise the internal temperature from 30 F to as much as 70 F above the temperature at which the concrete is placed. The rapid dissipation of heat to the surrounding air in beams, columns, walls or other masses of concrete less than 2 ft thick prevents the development of excessively high internal temperatures, but sudden drops in surface temperature of a destructive nature may occur in such thin sections.


Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then in Water at 72°F.
Made at 80°F. Placed in water immediately after molding
72°F.
50°F.
33°F.
Made at 80°F. ¼ d. Prelim. Storage at 80°F, then in water
Made at 80°F. 1d. Prelim. Storage at 80°F, then in water
Made at 80°F. 3d. Preliminary Storage at 80°F, then in water
Made at 50°F. 1d. Preliminary Storage at 50°F. then in water
Specimens placed in water immediately after molding and specimens given ¼ d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
0 4 8 12 16 20 24 28
Age at Test - days

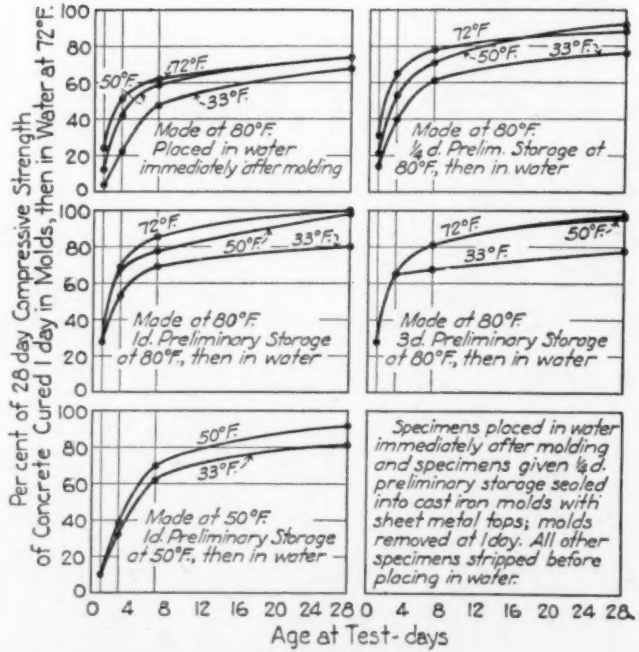

Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then in Water at 72°F.
Made at 80°F. Placed in water immediately after molding
72°F.
50°F.
33°F.
Made at 80°F. ¼ d. Prelim. Storage at 80°F, then in water
Made at 80°F. 1d. Preliminary Storage at 80°F, then in water
Made at 80°F. 3d. Preliminary Storage at 80°F, then in water
Made at 50°F. 1d. Preliminary Storage at 50°F, then in water
Specimens placed in water immediately after molding and specimens given ¼ d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
0 4 8 12 16 20 24 28
Age at Test - days

Fig. 3 (left)—Relative strength of concrete as influenced by temperature of storage water—normal cement
Fig. 4 (right)—Relative strength of concrete as influenced by temperature of storage water—high early strength cement

From Timms, A. G., and Withey, N. H., "Further Studies of Temperature Effects on Compressive Strength of Concrete", ACI JOURNAL, Nov.-Dec. 1934, *Proc.* V. 31, p. 165

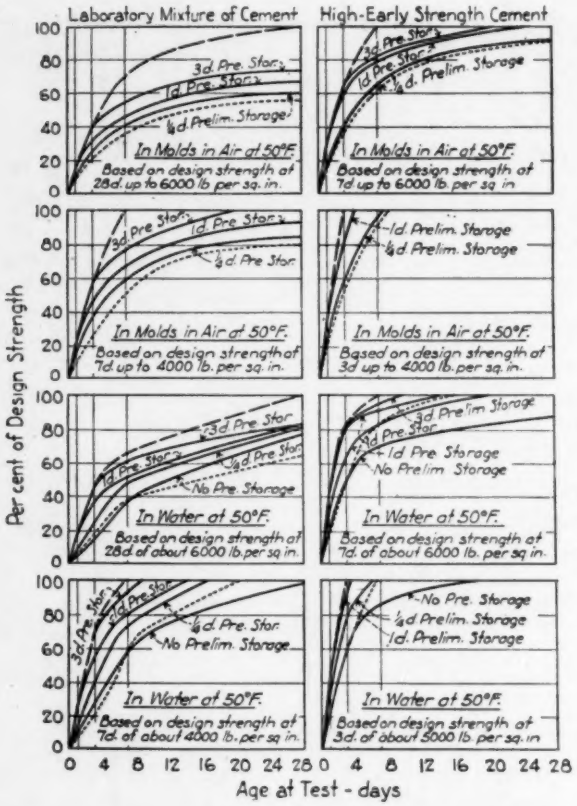

Laboratory Mixture of Cement
High-Early Strength Cement
Per cent of Design Strength
Age at Test - days
In Molds in Air at 50°F.
Based on design strength at 28d. up to 6000 lb. per sq. in.
Based on design strength at 7d up to 6000 lb. per sq. in.
Based on design strength of 7d up to 4000 lb. per sq. in.
Based on design strength at 3d up to 4000 lb. per sq. in.
In Water at 50°F.
Based on design strength at 28d. of about 6000 lb. per sq. in.
Based on design strength at 7d. of about 6000 lb. per sq. in.
Based on design strength at 7d. of about 4000 lb. per sq. in.
Based on design strength at 3d. of about 5000 lb. per sq. in.
3d Pre. Stor.
1d Pre. Stor.
½d Prelim. Storage
½d Pre. Stor.
1d Prelim. Storage
3d Prelim Storage
No Pre. Storage
No Prelim. Storage

Fig. 5—Strengths at different ages for concrete stored in air or in water at 50 F expressed as percentages of design strengths for wet curing at 70 F

Dash lines for concrete given 1 day preliminary storage in air, then moist at 70 F (top 4 diagrams), or in water at 72 F (bottom 4 diagrams).

Solid lines for concrete made and given preliminary storage in air at normal temperatures, then stored in air or water at 50 F.

Dotted lines for concrete made and given 1 day preliminary storage in air at 50 F, then stored in air or water at 50 F.

From Timms, A.G. and Withey, N. H., "Further Studies of Temperature Effects on Compressive Strength of Concrete", ACI JOURNAL, Nov.-Dec. 1934, *Proc.* V. 31, p. 165.

5. *Obtaining the degree of economy consistent with the design or intended use of the concrete*—In determining the winter concreting methods to be used on a particular job the fact should be kept in mind that most concrete structures are designed for a useful life of many years instead of the few weeks or months covered by ordinary construction operations or laboratory test periods. Satisfactory strength for 28-day cylinders is not enough if the structure shows frostbitten corners, dehydrated areas and cracking from overheating, all the result of inadequate protection and curing, or careless supervision. Similarly, early strength and good appearance of a structure achieved as a result of the use of excessive amounts of cement or calcium chloride will be of no avail if the concrete cracks badly in later years or develops disruptive internal expansion

as a result of chemical action. The last penny of construction economy should not be obtained by sacrificing durability.

Experience has shown, however, that the cost of adequate protection for cold weather concrete need not be excessive. Depending on the type of structure and the exposure, the cost of the concrete may be increased up to 10 percent over similar concrete placed under summer conditions. Since neglect of early frost protection may result in immediate destruction or permanent weakening of the concrete, it is apparent that adequate protection from low temperatures and the provision of proper curing conditions are very important factors on all concrete structures built during cold weather.

Summary

In the preceding paragraphs the nature of the cold-weather concreting problem has been set forth briefly to provide a background for a workable recommended practice. There has been no attempt to treat the subject exhaustively. The literature on cold-weather concreting is voluminous. Its extent in engineering periodicals since 1930 is evident from the attached list of selected references to which the constructor, the engineer and the specification writer are referred for additional information.

It is apparent that the best construction practice for thin sections such as buildings, pavements and structural sections less than 2 ft thick will be somewhat different from the best practice for mass concrete such as dams and large piers where the heat of hydration of the cement is a factor. In the first case, the problem is to keep the thin sections from freezing and to permit the concrete to harden and gain strength properly. In the second case, the problem is to keep the temperature change down and at the same time prevent local frost damage to corners and exposed faces.

In either case there is need for a factor of safety to take care of unexpected changes in weather and the small amount of careless execution of the work that is inevitable, even on well-managed jobs.

With all these things in mind, two sets of recommended practices for cold weather concreting were prepared, one for buildings, pavements and other thin sections, and one for mass concrete.

These recommended practices should be treated as general recommendations and not as detailed specifications covering every case of concreting in cold weather.

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Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions*

By JACOB PORTER FRANKEL†

SYNOPSIS

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

INTRODUCTION

The two methods of determining the tensile strength of concrete most quickly brought to mind are the simple tension and the beam flexure tests. The tension tests are costly and require much skill and labor—disadvantages that point to the flexure tests as the most convenient and least costly method. However, the modulus of rupture determined for beams has been found to be from one and one-half to more than two times the strength indicated by the most careful tension tests. In the past this discrepancy has been credited to the inapplicability of the assumption of a straight-line stress distribution in a beam of concrete. That this can account for a maximum difference of only 50 percent of the tension test strength (by assuming the rectangular stress distribution used in limit design and first assumed by Galileo) has led to a search for more cogent explanations of the discrepancy.

Additional tests have revealed other factors not adequately explained by classical theory; notable among these are the decrease in the modulus of rupture with increasing dimensions of the beams. However, the one

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consideration that, more than any other, led to the development of the statistical theories was that tests of brittle materials gave much less uniform results than the same tests of more ductile materials*. Whereas the classical theory uses only the mean value of the test results, the statistical theory is based on the distribution of test values about the mean.

As long ago as 1927 J. Tucker, Jr.,[1] explained the variation of the compressive strength of concrete with the size of the cylinders tested by assuming that the distribution of results of identical tests made on identical cylinders would be a "normal" one. The use of the normal curve in this regard was implicity questioned by W. Weibull in 1939[2], who, independently of Tucker, set up a statistical theory employing a simplified distribution curve that he found applicable to many brittle materials. Weibull's method had the added advantage of being handled analytically with relative ease, compared to the comprehensive tabulations that must be consulted when using a normal distribution. Since then, Tucker[3] has demonstrated quite clearly the adaptability of the statistical theory to the explanation of the variation of the modulus of rupture with the beam dimensions. Only Weibull's method allows an analytical derivation of the relative strengths in tension and bending.

It was felt by the writer that due to inherent similarities in structure of mortar and concrete, tests on mortar beams would offer an approximate idea of the behavior of the larger concrete specimens. Only bending tests were performed; the effect of size of specimen and type of loading are investigated herein.†

STATISTICAL THEORY‡

The probability of a *unit* of a given material safely withstanding a given stress is a function of the stress itself; *i.e.*, the larger the stress, the less the probability of successfully withstanding that stress. If the mean strength of many such units of a given material is determined, the probability that any *one* unit of the material can safely carry this stress is one-half; that is, the chances are equally great that it will be stronger or weaker than the mean.

If the probability of the occurrence of a set of n independent events are given by P_1, P_2, P_3 . . . . . P_n, respectively, then the probability that all the events will occur simultaneously is given by

$$[P_n] = P_1 \cdot P_2 \cdot P_3 \cdot \cdots \cdot P_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

If the probability of failure of a unit volume be designated by P_o, the

*The term *ductile material* denotes a material tested in such a manner that failure by flow precedes actual rupture.

†It is noted that Weibull has reported tests made on tension specimens of mortar made with standard German sand. The type and size of the specimens were not specified, nor were the data analyzed in full.

‡A complete list of symbols used is included in the appendix for convenient reference. All symbols are defined when they first occur.

probability of success of a unit volume is $1 - P_o$, and the probability of success of a specimen of volume V is

$$1 - P_V = (1 - P_o)^V \quad \ldots\ldots (2)$$

since there are V unit volumes in the volume V. That is, the probability of success at any particular stress, for a specimen of many units is less than that of one unit, since the more the units, the greater the chance of introducing a weak unit. True, the chances of having units stronger than the average are just as great, but in a brittle specimen subjected to a uniform stress the strength is determined by the weakest units.* Taking the logarithm (to the base e) of both sides of Eq. (2).

$$\ln (1 - P_v) = V \ln (1 - P_o) \quad \ldots\ldots (3)$$

Defining the *risk of rupture* of the specimen as

$$R = - V \ln (1 - P_o) \quad \ldots\ldots (4)$$

one obtains the fundamental equation of the statistical theory

$$P_V = 1 - e^{-R} \quad \ldots\ldots (5)$$

It may be noted that R is proportional to the volume and the quantity $\ln(1 - P_o)$ which, for materials of randomly distributed elements, is a function of the stress alone.

Accordingly, the risk of rupture of an infinitesimal volume, dV, is given by

$$dR = f(S)\, dV \quad \ldots\ldots (6)$$

where $f(S)$ denotes a function of stress alone. Integrating, the general expression for R becomes

$$R = \int f(S)\, dV \quad \ldots\ldots (7)$$

To determine the ratio of the maximum stresses the material may sustain under different states of stress one need only equate the values of R obtained from Eq. (7) when the appropriate values for $f(S)$ are substituted therein.

Thus the statistical theory gives a new significance to the meaning of relative strengths under different states, namely, the maximum stresses for which the risks of rupture are the same. The ultimate strength of a material may be designated as the mean of the test strengths of many specimens *of the same volume* under the same state of stress.

Weibull's method

Weibull found that $f(S)$ may be expressed, with a reasonable degree of accuracy, by the expression

$$f(S) = \left(\frac{S}{S_o}\right)^m \quad \ldots\ldots (8)$$

*No ductility is present to redistribute stress from weaker to stronger elements; a crack propagates at a very rapid rate.

where m is a material constant exemplifying the scatter of the data, S_o a particular stress constant and S the stress at the element dV.

For simple tension, S is constant throughout the specimen; the risk of rupture is given by

$$R_{Tension} = \left(\frac{S_{max}}{S_o}\right)^m V_T \quad \ldots\ldots (9)$$

For pure bending, S varies with the distance of the element dV from the neutral axis. For a beam of width b, length L, and depth $2c$

$$S = S_{max}\frac{y}{c} \quad \ldots\ldots (10)$$

where y is the distance of the element dV from the neutral axis. Since the probability of the specimen failing on the compression side is negligible compared to the probability of tensile failure, R for pure bending is given by

$$R_{Bending} = \frac{bL}{S_o^m}\int_0^c \left(\frac{S_m y}{c}\right)^m dy \quad \ldots\ldots (11)$$

$$= \left(\frac{S_m}{S_o}\right)^m \frac{V_B}{2(m+1)} \quad \ldots\ldots (12)$$

Equating the right sides of Eq. (9) and (12), and setting V_T equal to V_B one finds the ratio of the bending and tension strengths of the specimens of the same volume to be

$$\frac{S_{m(B)}}{S_{m(T)}} = \left[2(m+1)\right]^{\frac{1}{m}} \quad \ldots\ldots (13)$$

It may also be shown that the risk of rupture for center-point loading is given by

$$R_{cp} = \left(\frac{S_m}{S_o}\right)^m \frac{V_{cp}}{2(m+1)^2} \quad \ldots\ldots (14)$$

In practice it is difficult to obtain pure bending; to this end any symmetrical beam loading may be used, provided that the volume included between the loading points (region of constant moment) is sufficiently large so as to preclude failure elsewhere.

Tucker's method

Rather than specialize the form of $f(S)$ in the manner of Weibull, Tucker postulates that the distribution curve obtained would be identical to that describing the *normal law of errors*. The equation of this curve is

$$P = \frac{1}{a\sqrt{2\pi}}\int_{-\infty}^{(S-S_{avg})} e^{-\frac{(S-S_{avg})^2}{2a^2}}\, d\,(S-S_{avg}) \quad \ldots\ldots (15)$$

where P is the probability of failure at a stress equal to or less than S, e is the base of the natural logarithms, and a is the standard deviation of the test results. Since S cannot be less than zero, the lower limit of the intergral in Eq. (15) may be made zero, provided the origin of the curve is shifted an amount S_{avg} to the left, obtaining

$$P = \frac{1}{a\sqrt{2\pi}} \int_0^S e^{-\frac{S^2}{2a^2}} dS \quad \ldots\ldots (16)$$

The integration is simplified by expanding the integrand as a power series, yielding

$$P = 1 - \frac{e^{-t^2}}{t\sqrt{\pi}} \left(1 - \frac{1}{2t^2} + \frac{3}{4t^4} - \ldots\right) \quad \ldots\ldots (17)$$

where t replaces $\frac{S}{a\sqrt{2}}$. For all but the smallest values of S, Eq. (17) reduces to the form

$$P = 1 - C_1 e^{-C_2 S^2} \quad \ldots\ldots (18)$$

Returning to Eq. (5), (7) and (9), we find that Weibull's method gives, for tension,

$$P = 1 - e^{-V\left(\frac{S}{S_o}\right)^m} \quad \ldots\ldots (19)$$

Comparing this with Eq. (18) one finds that Tucker's hypothesis is based on an equation of exactly the same form as that adopted by Weibull, with the added specialization that $m = 2$.

EXPERIMENTAL METHODS

Weibull has pointed out that because of care taken in the preparation of all specimens and the necessary handling prior to the actual tests the lower limit of the recorded strengths cannot be as low as zero; rather it is assumed that the least value of S possible is a definite value S_l, which is lower than any test strength by an amount that decreases with increasing numbers of test specimens. To this end a new variable $(S - S_l)$ replaces the variable in all the equations previously established. From Eq. (5), (12), and (13) we may establish the basic probability equation for a brittle material under any state of stress as

$$P = 1 - e^{-kV\left(\frac{S - S_l}{S_o}\right)^m} \quad \ldots\ldots (20)$$

Rearranging and twice taking the logarithm of this equation, first to the base e, then to the base 10 one obtains

$$\log \ln \left(\frac{1}{1-P}\right) = m \log (S - S_l) + \log kV \quad \ldots\ldots (21)$$

It may be noted that if log ln $\left(\frac{1}{1-P}\right)$ be plotted against log $(S-S_l)$ a straight line should result, the slope of which is m.

In an actual case the value of S_l may be determined by trial, that value yielding the straightest line being chosen as the most probable value.

The simplest way to determine P from a set of test values* is as follows:

1. List the test strengths in order of increasing strength, neglecting duplicate values.

2. Number all strengths ordinately, *i. e.*, lowest is 1, next is 2, etc., until the number of the last strength is n, the number of specimens tested. If more than one specimen failed at the same strength, the strength will have that many numbers.

3. To each strength assign a rank equal to 0.5 less than the number or mean of the numbers assigned in step 2.

4. P then = rank/n.

The factor k in Eq. (20) is a function of m alone; typical values are shown in Fig. 1.

*See Table 7, p. 32, for an example of this operation.

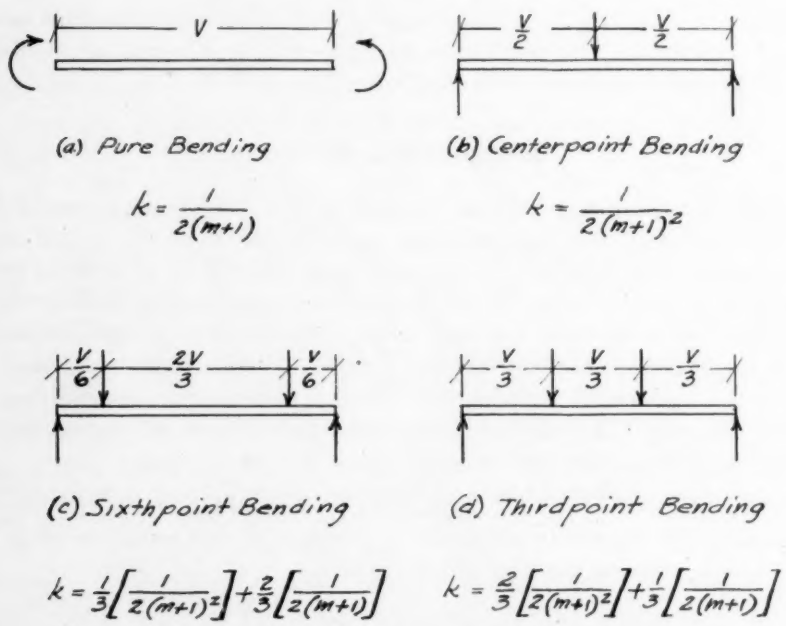


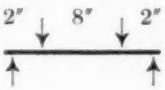
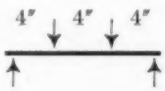
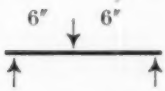
Fig. 1—Expressions for the factor K of Eq. (20)

Having determined S_t, and m, k may be computed for the particular state of stress to which the specimens were subjected, and S_o determined by assuming $S_o = S - S_t$, solving Eq. (20) for P, and then consulting the table for the stress corresponding to this probability of failure. S_o is then equal to this stress less the previously determined S_t.

Testing

Tested at the University of California were three series of portland cement mortar beams, listed in Table 1.

TABLE 1—SUMMARY OF TESTS

Series	Number of Beams	Type of Loading
A	30	
B	35	
C	34	

All the beams were 1 x 1 x 12 in. Calaveras early hardening plastic cement and standard 20 to 30 mesh Ottawa sand mixed with water in accordance with A.S.T.M. Standard C77-40 comprised the mortar. The mix was compacted in wooden molds 1 in. wide, 1 in. deep and 25 in. long, with aluminum base plates. Compaction was by thumb, in the same manner as for tensile briquets. All the specimens were cured for 7 days in a fog room held at 70 F and 100 percent relative humidity. Immediately before testing the beams were removed from the fog room and each was sawed into two 12½-in. lengths.

The tests were performed on a Toledo scale which served to measure the applied loads within 1 lb; since the readings ranged from 49 to 109 lb, it was felt that two-figure accuracy must suffice. The loads were applied through a spreader bar and rollers on bearing plates; the reactions consisted of a spherical bearing at one end, and a roller bearing at the other end.

TABLE 2—SUMMARY OF TEST RESULTS

Item	Series A	Series B	Series C	Weighted mean
$\bar{S}_m$ (psi)	540	770	1000	—
S_l (psi)	340	520	720	—
S_o (psi)	230	280	220	240
m	2.6	2.8	3.4	2.9

TABLE 3—COMPARISON OF THEORETICAL AND EXPERIMENTAL MEAN STRENGTHS

Based on $\bar{S}_{mB}$ = 770 psi

Series	Theoretical mean strength, psi	Experimental mean strength, psi
A	550	540
B	770	770
C	1040	1000

Results

Table 2 contains the results of the statistical analyses of the test results.*

Taking the mean strength of the beams in Series B, 770 psi, as a base, we may determine theoretical values of the mean strengths of Series A and C by equating the theoretical expressions for the risk of rupture [similar to Eq. (12) and (14)]. Thus

$$\bar{S}_{mA} = 340 + \left(\frac{k_B}{k_A}\right)^{\frac{1}{m}} (\bar{S}_{mB} - S_{lB}) \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)\dagger$$

and

$$\bar{S}_{mC} = 720 + \left(\frac{k_B}{k_C}\right)^{\frac{1}{m}} (\bar{S}_{mB} - S_{lB}) \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)\dagger$$

Using $m = 2.9$, the expressions for k from Fig. 1 and the values of S_l from Table 2, Eq. (22) and (23) yield the comparison shown in Table 3.

CONCLUSIONS

1. The statistical theory developed by W. Weibull is seen to apply to the strength of portland cement mortar under the different types of bending investigated.

*For the convenience of those who wish to further analyze the test results, details and sample calculation are given in the appendix, Tables 4, 5, 6 and 7. Fig. 2 in appendix contains plots of Eq. (21) for the three series.

†The numerical values shown in Eq. (22) and (23) are the values of S_l taken from Table 2.

2. The strength of portland cement mortar is dependent on the volume of the specimen, and has the following relative values* for the mortar tested:

Relative volume	Relative strength
1	1.00
2	0.78
5	0.57
10	0.45
100	0.20

3. The strength of a member of brittle material depends on the state of stress to which the member is subjected; the less the volume subjected to the greatest stress, the higher the overall strength of the member. Simple tension, pure bending and center-point bending exhibit strengths which are increasingly greater, in the order given.

4. The similarity in structure between mortar and plain concrete would seem to make the statistical theory particularly applicable to similar problems in concrete. Further tests of concrete beams are necessary to establish the m values for different mixes.

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APPENDIX

Notation

The symbols are listed in the order in which they first appeared.

P = Probability of failure.
P_o = Probability of failure of a unit volume.
P_V = Probability of failure of a volume, V.

*Computed from the relationship $\frac{S_2}{S_1} = \left(\frac{V_1}{V_2}\right)^{\frac{1}{2.9}}$

S = Stress.
S_m = Maximum stress in a member (modulus of rupture of a beam).
$\overline{S_m}$ = Mean modulus of rupture of a set of specimens.
R = Risk of rupture = $-V \ln (1\text{-}P_o)$.
m, S_o = Constants for any given material.
b = Beam width.
c = One-half beam depth.
L = Beam length.
y = Distance of elemental unit from neutral axis in a beam.
a = Standard deviation of a set of test results.
k = Function of m, constant for any given type of loading.
S_l = Lowest possible strength in a set of specimens under a given type of loading.
n = Number of specimens tested in a single series.

Individual test results

Tables 4, 5 and 6 give details of strength tests conducted at the University of California. Fig. 2 contains plots of Eq. (21) for the three series.

TABLE 4—TEST STRENGTHS FOR THE 30 BEAMS OF SERIES A, IN ORDER OF INCREASING MAGNITUDE

Sixth-point loading, span 12 in.

Number	Ultimate stress, psi	Number	Ultimate stress, psi	Number	Ultimate stress, psi
1	400	11	500	21	580
2	400	12	510	22	580
3	410	13	520	23	590
4	460	14	530	24	620
5	460	15	540	25	620
6	460	16	540	26	620
7	470	17	550	27	640
8	480	18	560	28	650
9	490	19	560	29	650
10	490	20	580	30	650

TABLE 5—TEST STRENGTHS OF THE 35 BEAMS OF SERIES B, IN ORDER OF INCREASING MAGNITUDE

Third-point loading, span 12 in.

Number	Ultimate stress, psi	Number	Ultimate stress, psi	Number	Ultimate stress, psi
1	580	13	730	25	800
2	610	14	760	26	830
3	610	15	760	27	830
4	620	16	770	28	830
5	630	17	770	29	850
6	670	18	780	30	860
7	680	19	780	31	860
8	680	20	780	32	880
9	710	21	780	33	900
10	710	22	780	34	920
11	730	23	800	35	950
12	730	24	800		

Sample calculation—Series B

To find S_o, let $S - S_l = S_o$. For series B,

$S_o = S - 520$

For this stress

$$P = 1 - e^{-\frac{12}{15.4}(1)^m} = 0.65$$

Consulting Table 7 (p. 32), the stress corresponding to

P = 0.65 is found to be

S = 800 psi, to two significant figures.

S_o = 800 − 520 = 280 psi.

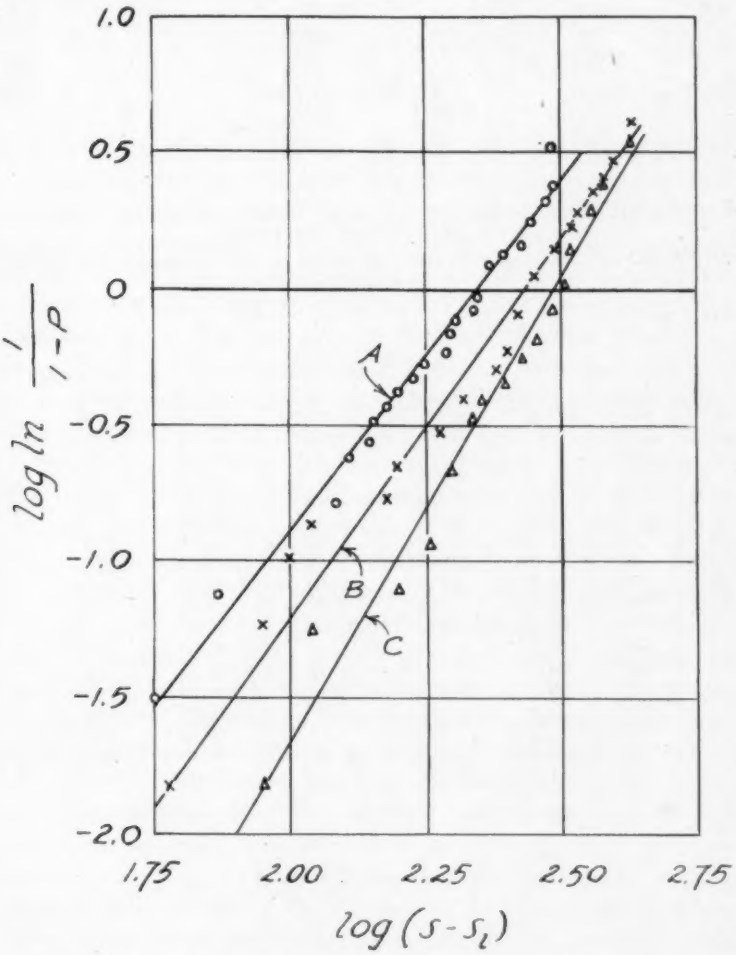

Fig. 2—Statistical plots of bending test data, portland cement mortar, age 7 days Volume of specimen is 12 cu in.

A—Sixth-point loading
B—Third-point loading
C—Center-point loading

TABLE 6—TEST STRENGTHS OF THE 34 BEAMS OF SERIES C, IN ORDER OF INCREASING MAGNITUDE

Center-point loading, span 12 in.

Number	Ultimate stress, psi	Number	Ultimate stress, psi	Number	Ultimate stress, psi
1	810	13	970	25	1060
2	830	14	990	26	1060
3	880	15	990	27	1060
4	900	16	990	28	1060
5	920	17	1010	29	1080
6	920	18	1010	30	1080
7	920	19	1030	31	1100
8	920	20	1030	32	1100
9	920	21	1030	33	1150
10	940	22	1040	34	1150
11	950	23	1040		
12	950	24	1060		

TABLE 7—REDUCTION OF DATA—FOURTH TRIAL OF SERIES B USING $S-520$ AS INDEPENDENT VARIABLE

($S-340$, $S-500$, $S-540$, had all previously failed to yield a straight line on log ln vs. log plot)

n	S	Rank	P	$\frac{1}{1-P}$	$\log \ln \frac{1}{1-P}$	$S-520$	$\log S-520$
1	580	0.5	0.0143	1.015	−1.83	60	1.78
2–3	610	2.0	0.0572	1.060	−1.23	90	1.95
4	620	3.5	0.100	1.11	−0.98	100	2.00
5	630	4.5	0.128	1.148	−0.86	110	2.04
6	670	5.5	0.157	1.178	−0.79	150	2.18
7–8	680	7.0	0.200	1.25	−0.65	160	2.20
9–10	710	9.0	0.257	1.35	−0.52	190	2.28
11–13	730	11.5	0.328	1.49	−0.40	210	2.32
14–15	760	14.0	0.400	1.67	−0.29	240	2.38
16–17	770	16.0	0.457	1.84	−0.22	250	2.40
18–22	780	19.5	0.556	2.26	−0.09	260	2.42
23–25	800	23.5	0.672	3.05	0.05	280	2.45
26–28	830	26.5	0.757	4.12	0.15	310	2.49
29	850	28.5	0.814	5.38	0.23	330	2.52
30–31	860	30.0	0.856	6.95	0.29	340	2.53
32	880	31.5	0.900	10.0	0.36	360	2.56
33	900	32.5	0.929	14.1	0.42	380	2.58
34	920	33.5	0.957	23.3	0.48	400	2.60
35	950	34.5	0.985	66.7	0.62	430	2.63
Mean	770						

Discussion of a paper by J. P. Frankel:

Relative Strengths of Portlant Cement Mortar in Bending Under Various Loading Conditions*

By J. TUCKER, Jr. and AUTHOR

By J. TUCKER, Jr.†

Mr. Frankel's analysis is based on the statistical theory of the strength of materials, a relatively new subject in which more and more interest is being shown. The theory predicts larger proportional changes in strength the smaller the specimen[1], hence is particularly adapted to small test specimens. The differences in strength of full size structural beams would be much smaller and therefore Mr. Frankel's statement that tests on mortar beams would offer an approximate behavior of large concrete specimens should not be accepted without considerable reservation. There is also the additional variable of size of largest aggregate present as it has been shown that the strength dispersion increases with size of aggregate[2].

Mr. Frankel makes a statement concerning my work in statistical strength theory which may give an erroneous impression of my underlying philosophy. He states that I assumed that the distribution of strength of concrete cylinders would be normal. This is not correct. I analyzed[2] the strengths of a number of groups of cylinders and concluded that the strength frequency curve was symmetrical and that "the data strongly indicate the normal curves." In my presentation[2] of the equivalent of Mr. Frankel's Eq. (2) I state "for the normal curve . . . ." The normal curve was selected because all data secured in the paper approximated the normal form. The only essential in my philosophy is the writer's Eq. (2). The probabilities in that equation are the probabilities from the strength frequency curve of the material, whether normal or decidedly differing from normal. I have always used the normal form because the data appeared to be of the normal type and because tables of probabilities are readily available for the normal form. It can be added that by the methods which have been used, satisfactory results have always been obtained.

*ACI JOURNAL, Sept. 1948, *Proc.* V. 45, p. 21. Disc. 45-2 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45.

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The Weibull method of treating data has a major weakness which is well brought out by Mr. Frankel's paper. This weakness consists in the treatment of all data as if differences in the strength of specimens of diverse dimensions is caused solely by the differences in the dimensions of the specimens. Any systematic error would cause the difference in strength between specimens of two dimensions to be considered as due to the differences in the dimensions. As an illustration suppose we have two sets of mortar beams, one tested at third points, the other at sixth points, both on equal spans. Let us further suppose that the third-point beams were tested in the position as cast, the other set in the inverted position. Because of the water rise, the top fiber would be weaker, hence the beams tested at third point should be stronger. If we make a further assumption that the strength dispersion is so small as to be negligible, then the difference in strength caused by the difference in loaded length would be also negligible. However, Weibull's methods of computation would quite erroneously indicate that the difference in strength which we obtained was due to the difference in dimensions.

The reduction in the strength of the sixth-point loaded beams below the strength of the third-point loaded beams found by Frankel is about four and a half times as large as the reduction due solely to the statistical effect caused by change in the size of the loaded length. The latter value may be computed by the use of Eq. (2) as follows.

It may safely be assumed that the beams with third-point loading will all fail within the middle third length of 4 in. and that the beams with sixth-point loading will all fail within the length between loading points, or 8 in. The latter beams may then be considered as being composed of two specimens each 4 in. long or of two specimens, each the same length as the third-point beams. The 8-in. specimens, considered as a combination of two 4-in. specimens in series, will have the strength of the *weaker* of the two 4-in. specimens of which it is composed. Statistical methods permit us to compute the average strength of the weaker specimen in a series of groups of two specimens. Tippett[3] has given tables for such values, computed on the basis of a normal strength distribution of the individual (in this case the 4-in. specimen). According to Tippett the mean strength of the lowest in a group of two is

$$M_2 = M_1 - 0.59\sigma_1$$

where M_1 is the mean strength of individuals, and

σ_1 is the standard deviation of the strength of the individual specimens.

The standard deviation of the strength of modulus of rupture of the beams loaded at third points is 92 psi; the mean strength is 765 psi. Therefore, the computed modulus of rupture of the beams loaded at sixth points would be $765-(0.59 \times 92)=713.5$ psi. This is considerably less

reduction in strength than Frankel found in his tests and shows that there was a very large systematic error somewhere in those tests. It is to be again noted that by Weibull's method this systematic error remains undiscovered, and, in fact, unknown.

Disciples of Weibull would probably object to the foregoing computations on the basis that the actual strength distribution of the beams was not normal. To answer any such argument the following test was made, to which there can be no such objection, and in which the actual strength distribution of the third-point beams was used.

The strength of each third-point beam was written on a small square slip of paper. The 35 slips obtained were placed in an urn (water glass to the nonstatistician) and well stirred. The papers were drawn out two at a time (one after another). The pairs of values were recorded, the slips returned to the urn, well mixed and drawings again made. Thirty drawings of two slips were made. The lowest of the values in each set of two was recorded. These values should simulate the strengths of the 30 sixth-point beams if length were the only variable and the mortar in all bars came from the same statistical universe.

The mean value of 710 psi was obtained from our drawings. To verify results and show that the value was not obtained by accident the whole set of drawings was repeated. This time a mean value of 725 psi was obtained. The average of these two values is 717.5 psi, sufficiently close to the theoretical value of 713.5 secured by Tippett's tables to confirm the validity.

A simple examination of the test data itself will convince one that a definite systematic error is present. The strength of the third-point specimens vary from 580 to 920 psi. That is, 100 percent of the samples have strengths between these values. Therefore, it is known that a value of close to 100 percent of the specimens of the theoretical universe from which the sample was selected will be within the same range. In other words, the percentage of the universe with strengths below 580 psi would be very small and negligible. However, we find in Frankel's sixth-point group that 19 of the 30 specimens had strengths less than 580 psi, a finding to make one pause and reflect.

No explanation is offered for the large systematic error, which may have been due to a number of causes, and which would be unknown to anyone not present at the testing. The stresses produced by short spans or high loadings, as theoretically investigated by Karman and Seewald[4] should be examined by Mr. Frankel to determine their influence, which may at least in part explain the discrepancy. It is not thought, however, that these stresses would be nearly large enough to explain that discrepancy.

There are other criticisms which may be made of Weibull's method, even though they have no direct bearing on Mr. Frankel's application of the method. For one thing, his selection of the lowest possible value for the particular statistical universe. This value has to the writer neither mathematical or physical significance. Another criticism may be made of Weibull's method of using specimens which vary in both longitudinal and lateral dimensions. This would be acceptable provided that the same probability would be valid for links in series and in parallel. Experimental as well as theoretical considerations have shown that the two probabilities are not equal, and therefore simultaneous longitudinal and lateral variation in dimensions should be prohibited.

However, the foregoing has no connection with the present series of tests. Fortunately, Mr. Frankel has varied his specimens only in longitudinal dimensions which has permitted a true analysis and brought out the systematic error. This would not have been possible had both lateral and longitudinal dimensions been varied.

Weibull also has assumed that the stress is proportional to the distance from the neutral axis, an assumption from experimental findings on concretes and mortars. The writer has made a study of the stresses in beams present during test[5], which differ greatly from the values obtained on this assumption.

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AUTHOR'S CLOSURE

Mr. Tucker's discussion serves to bring out both the strength and weakness of the Weibull statistical theory.

In ascertaining the statistical variation of the strength of materials the basic approach is a negative one. The calculations are based on the distribution of weaknesses, the assumption being that the greater the volume the greater the number of weaknesses, or flaws, that may be present. This in itself seems valid but its importance is outweighed by the now accepted view that only the surface flaws are critical, those situated within the material not influencing the failure.

This point of view was developed and strengthened by the work of Joffé with rock salt, Smekal with glass, and Orowan with mica, among others. An excellent symposium of their views is contained in *Papers and Discussions*, V. II, International Conference on Physics, published by the Physical Society, 1935.

If one accepts this premise that only the surface flaws are important, then the equation should show a dependence of strength upon surface area, rather than volume. Thus Mr. Tucker's observations regarding the strength of links in series compared to those in parallel are well taken. Again it must be pointed out, however, that in the tests described in the paper, since the cross section of all specimens was the same, the volume and surface area were always in the same proportion thus eliminating these objections from the data at hand.

On the other hand, the writer believes that what Mr. Tucker considers the weakness of the Weibull theory is really its strength. The basic difference in the application of Weibull's and Tucker's methods is in the nature of the *universe* of elements from which the specimens are formed.

Mr. Tucker must assume that the smallest beam tested contains enough elements to be representative of the entire universe of elements. If that were so, then he is quite justified in considering a 8-in. beam as two 4-in. beams tested simultaneously.

Weibull's theory makes no such postulation; in effect it says: We do not know the size of the *element*, hence we can not suppose that any one specimen containing many elements is representative of the entire universe. If we have a larger specimen than the one first tested, it is reasonable to suppose that the second specimen is more representative of the universe than the first, and should thus contain some weaker elements than the first. Presumably the entire universe contains elements whose strength varies from the smallest that permits handling up to the theoretical cohesive strengths. Thus, it is to be expected that the larger the specimen the more representative it becomes and the lower should be its *minimum* strength.* In this light, the term S_l in Eq. (20) assumes a rightful place of physical significance.

Nevertheless, one still must examine the tests to determine if possible any systematic errors. Careful reexamination of the test procedures has failed to reveal any (with one reservation). The number of beams tested may have been too small to permit accurate handling of the data. This could be readily checked by repetition of the experiments with facilities and time that would permit many more tests, neither of which are at present available to the writer.

*It must be obvious that such tests as Tucker's "water-glass selection" can not possibly yield any strength lower than the least strength of the four-inch beams.

The local stresses investigated by Karman and Seewald can have only negligible effect since in the sixth-point loading tests 24 of the 30 beams tested broke at a distance greater than 1 in. from the point of application of load. St. Venant's principle, of course, allows one to neglect stress deviations due to local effects if one focuses his attention on locations a greater distance from the singularity than the depth of the beam, in this case 1 in.

As to whether the results of small beam tests are indicative of the behavior of large beams one may only conjecture. It is true, as Mr. Tucker says, that "the differences in strength of full size beams would be much smaller" provided that the beams were made from the same mortar. But since large concrete beams are made with larger aggregates, the effect of increased size is offset by the fact that the particles comprising the material are of the same relative size as those of the mortar beams. Recent work of E. Orowan in England has quantitatively established the importance of grain size on strength.

The writer is indebted to Mr. Tucker for his careful analysis of the paper, and ventures to hope that the discussions have served to bring to the attention of researchers in the field the usefulness of the statistical theories of the strength of materials.

Terrazzo as Affected by Cleaning Materials*

By D. W. KESSLER†

SYNOPSIS

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

INTRODUCTION

Because of the extensive use of terrazzo as a flooring material, both for its decorative and service value, it is important in its maintenance to guard against the use of cleaning preparations that may injure the surface. For this reason an investigation of the action of commonly used preparations in simulated cleaning cycles was undertaken in 1944 at the National Bureau of Standards, and, although the work is still in progress, certain results are sufficiently conclusive to be made available.

MATERIALS AND PROCEDURE

Aggregates

Seventy-six aggregates‡ obtained from the producers through the cooperation of the National Terrazzo and Mosaic Association, were used in this study. They included practically all of the domestic aggregates that have been widely used in terrazzo, and are believed to be repre-

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†Chief, Building Stone Section, National Bureau of Standards, Washington, D. C.

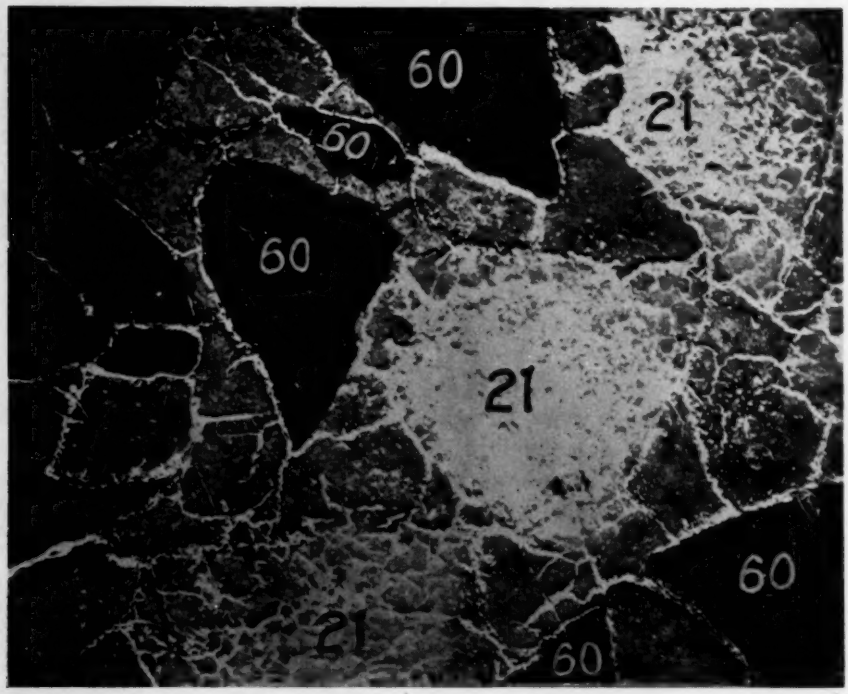
‡Other samples of these aggregates were used in a previous study reported in National Bureau of Standards *Building Materials and Structures Report* BMS98, "Physical Properties of Terrazzo Aggregates." This report is available at the U. S. Government Printing Office, Washington 25, D. C., 15 cents per copy.

sentative of materials now available. These marble samples, supplied by about 30 producers, were classed as calcites, magnesian marbles, dolomites and serpentines. The colors included white, black, various shades of red, yellow, green, blue and brown. Some of the samples were variegated sufficiently to produce beautiful color combinations with one aggregate.

Disk preparation

Since the purpose of the investigation was to test the aggregates under conditions similar to those in actual use, the samples were made into terrazzo disks. The specimens, carefully prepared from a sample of normal gray portland cement in the usual proportion of one part cement to two parts terrazzo chips, were cured for 3 months in damp storage before being surfaced. Using a special grinding device, each specimen was ground to form a dish; that is, a rim about $\frac{1}{16}$ in. higher than the finished surface was left at the edge. In the grinding process No. 80 silicon carbide was used, followed with No. 303 optical emery, and a final buffing with No. 500 silicon carbide to reveal the true colors. Before

Fig. 1—Photomicrograph (7½X) of terrazzo after 400 cleaning cycles with soda ash solutions. Half of the specimen disintegrated. The red aggregate (No. 60) remained in good condition while aggregate 21 (a travertine) was injured.



the final grinding, bubble holes were filled with a stiff grout made of the same cement mix. By the time the tests were started the specimens were about 6 months old.

Detergents

For the original test series two detergents were used, namely soda ash and trisodium phosphate. The first of these constitutes a part of many trade cleaning products often used on terrazzo. Trisodium phosphate is used either alone or in combination with other detergents. During the progress of the test when it was found that both types of cleaner caused harmful results, a synthetic sulfonate solution was included, since such preparations are being used to a considerable extent and little information is available as to whether or not they are injurious.

The detergents were used in the recommended* solution strengths of soda ash ¾ oz, trisodium phosphate ½ oz and synthetic sulfonate ¼ oz to the gallon of water. Burettes held in a specially constructed, hinged support were employed for applying the solutions (3 cc to each specimen) giving a depth of less than $\frac{1}{16}$ in.

Testing cycle

A cycle consisted of the application of the cleaning solution, rinsing after 30 minutes, followed by at least 6 hours of drying in an oven at 105 C. Because two cycles were completed each day the drying period varied, the specimens remaining in the oven overnight and over weekends.

OBSERVATIONS

According to these tests soda ash is more rapid in its destructive action on terrazzo than trisodium phosphate. These detergents bring about destruction in different ways. Thus far none of the specimens treated with the synthetic sulfonate have shown any definite signs of injury.

Sodium carbonate effect

Sodium carbonate crystals evidently form between the chips and matrix and in the pores of the matrix (cement paste) and aggregate. The expansive action of the crystals is severe and causes disintegration of both the aggregate and the matrix. Most marbles are more resistant than the matrix to this action, but a few show serious effects before disintegration of the terrazzo as a whole. Fig. 1 and 2 illustrate the destructive action of soda ash on terrazzo. The more resistant aggregates, 19 and 60, remained in good condition while aggregates 21 and 69 were injured. The effect seems to be the same as that caused by other efflorescent salts and is frequently evident on most types of masonry.

The greatest effect on the colors of aggregate was due to scaling of the surface, because, when the smooth surface of a colored aggregate is

*National Bureau of Standards *Letter Circular* LC764, "The Care of Floors".

removed, the color value is practically lost. In the more resistant aggregates where scaling did not occur the colors were not appreciably changed.

The first failure in the group of specimens treated with soda ash solutions occurred after 280 cycles, and all failed at less than 570 cycles. The average number of cycles for this group was 480. Fig. 3 shows some of the disintegrating disks.

Synthetic sulfonate effect

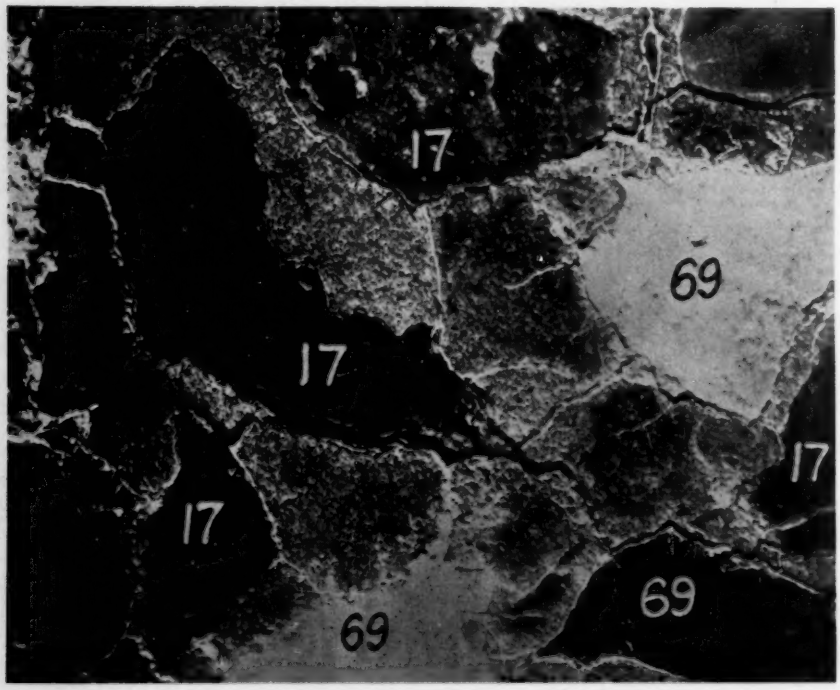
Specimens treated with the synthetic sulfonate solutions have undergone 850 cycles and no failures have occurred in this series to date.

Trisodium phosphate effect

The first failure in the trisodium phosphate group occurred at 560 cycles and over half of the specimens were still in a serviceable condition after 1000 cycles.

Although trisodium phosphate caused some efflorescence, this alone did not seem to be destructive. Most of the darker colored aggregates showed enough efflorescence to mar the colors. The main destructive action of the trisodium phosphate was apparently due to its part in the

Fig. 2—Photomicrograph (7½X) of terrazzo after 478 cleaning cycles with soda ash solution. The greatest injury was to the matrix but the variegated aggregate 69 shows surface scaling. The green aggregate (No. 17) was not perceptibly injured.



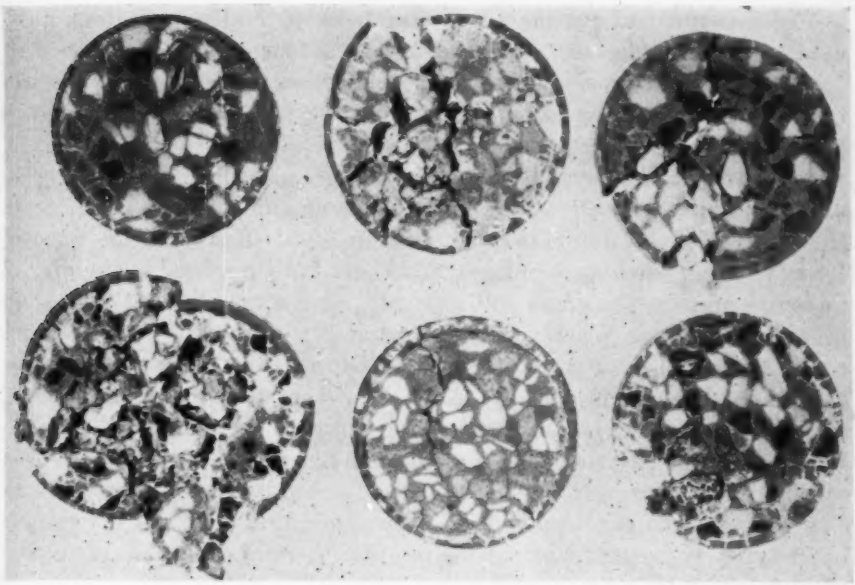


Fig. 3—Terrazzo specimens disintegrated by successive cleanings with soda ash solution Note that the action occurred mainly in the cement matrix which is weaker and much more porous than the marble chips.

formation of a hard veining material. This material formed wherever there was a space, such as a shrinkage crack or poor contact between chips and matrix. Evidently it started to form early in the tests before the veins became visible.

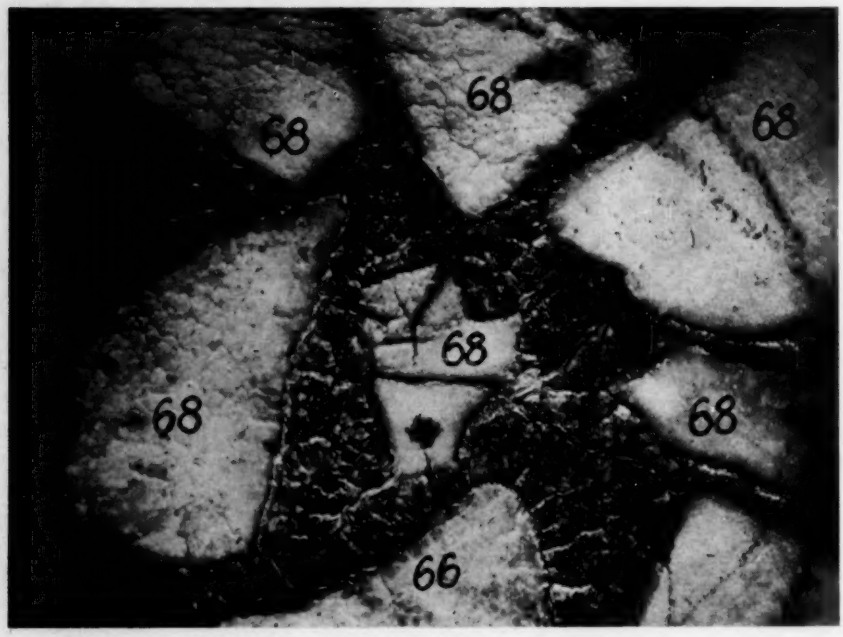
Penetration tests and the effect of veining

During the first few months, tests were made at intervals to determine if the specimens were becoming more porous. This was done by measuring the rates at which water penetrated. When the specimens were removed from the oven and cooled, 3 cc of water was placed on each and the time noted for complete absorption. By testing small groups of specimens it was possible to determine the rates of penetration with sufficient accuracy. For the trisodium phosphate group of specimens it was found that the time required for complete penetration increased greatly with the number of cleaning cycles. This was attributed to the sealing of minute cracks. Before 200 cycles were completed, the veining matter became visible and it continued to build up until small ridges were formed. These veins frequently formed between the chips and matrix as shown in Fig. 4 and 5. At the present time (about 1000 cycles) most of the marble chips are entirely surrounded by veining matter. At about 500 cycles it was noted that veinlets were forming between crystals in some of the aggregates. This is illustrated in Fig. 4.

The frequency of continuous veining between matrix and chips indicates a tendency for the veining to act as a "spear head" and grow in length. The ability of the veining to pierce aggregates is shown in the center of Fig. 4. The same action has been noted at masonry joints particularly in marble masonry, where several veins cross one joint, penetrate the adjacent stone and start cracks. This condition has been observed only where there is a considerable amount of dampness.

A theory for this action is proposed as follows: If a crack has formed for any reason, such as shrinkage, and there is a considerable amount of moisture present, the crack will start filling with soluble matter present in the masonry. Wherever the most water evaporates, usually at cracks, there will be greater amounts of solid matter deposited. In time the cracks become practically sealed near the surface and the water seeks other places to escape, for example through the pores of the stone. When this happens the soluble matter builds up in the surface pores causing a wedging action which may be accentuated by temperature changes. Marble is probably more susceptible to this action than some other types of stone because of the peculiar thermal expansion of calcite crystals, that is, positive along one axis and negative along another. However, other types of masonry are sometimes affected in the same way.

Fig. 4—Photomicrograph (7½X) of terrazzo after 1000 cleanings with trisodium phosphate solution. This shows a typical illustration of the veining matter and its tendency to pierce the marble chips. Aggregate 66 is somewhat more resistant to vein formations than 68.

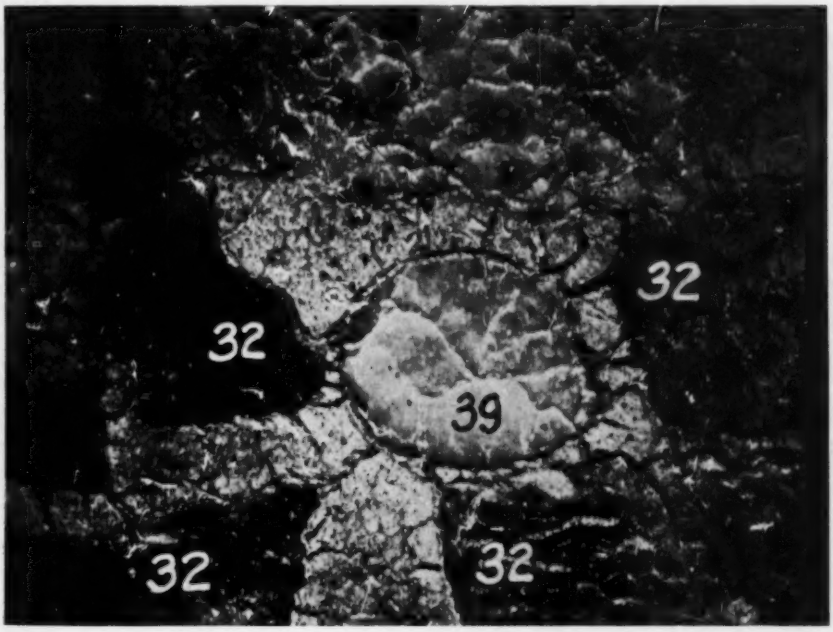


No serious attempt has been made, so far, to identify the composition of the veining matter in the terrazzo specimens. It appears to be of the same nature as that often found in masonry structures. Many such samples collected by the author have been found to be of a travertine nature, made up mainly of calcium carbonate. It is possible that the phosphate causes an accelerated action or forms a part of the veining. However, the veining in the specimens is not appreciably soluble in water.

Aggregate stability

Some positive indications were obtained in regard to the stability of the different aggregates when exposed to the severe conditions of the test. The white marbles proved to be less resistant, apparently because they are more completely crystallized. The salts or veining matter worked between the crystals and tended to break up the marble into small fragments. The temperature cycles seem to hasten the destruction in two ways: (1) Because of the positive and negative coefficients of expansion of calcite crystals, the heating and cooling cause stresses and slippage between the crystals and reduce the cohesion. (2) The inter-crystalline movements facilitate the formation of salt crystals within the pores. It has been proven that marbles undergo permanent expansion when exposed to repeated temperature cycles, and the amount

Fig. 5—Photomicrograph (7½X) of terrazzo after 1000 cleanings with trisodium phosphate. Veining is well developed in the matrix and in aggregate 39 while aggregate 32 is little affected.



of growth is greater if a weak salt solution (1 part in 500) is allowed to penetrate while at the high temperature.

The dense, fine-grained, colored marbles proved to be highly resistant to the destructive action. This group includes the black, red and most of the green aggregates. The reds showed some indication that their stability increases with intensity of color. In general the pinks were little more resistant than the whites. One red aggregate, with white spots, disintegrated at the white parts and remained sound in the red portions. Dark greens were more resistant than light and stability of the yellow and blue aggregates seems to depend on the crystalline nature.

When veining forms, as shown in Fig. 4 and 5, it is apparent that an overall expansion occurs because the veins have appreciable width. They form in many places where there was originally no space. No provision was made in the test to measure dimensional changes, but by calipering a considerable number of the specimens in the trisodium phosphate group after the veining matter had formed, it was possible to gain some idea of the expansion by comparing the average size of the specimen with the average diameter of the molds. These measurements indicated that the average diameter of the specimens was about 1.4 percent greater than that of the molds. Actually the "growth" was greater since some shrinkage occurred in the specimens in curing.

CONCLUSIONS

Because these Bureau of Standards tests were made under much more severe conditions than those of actual cleaning of terrazzo floors, it is impossible to draw definite correlations between the two. Although the strength of cleaning solutions was the same as those commonly used, a greater amount of solution was applied than would be used in the normal maintenance process. Furthermore, the temperature cycle was extreme in order to accelerate the crystallization of the salts. Nevertheless, it is considered that the following conclusions are justified:
1. Trisodium phosphate is less injurious to terrazzo than soda ash.
2. Present information on a synthetic sulfonate indicates that it is relatively free from injurious action. The results also seem to indicate the relative stability of the various aggregates.

In general, terrazzo floors are very durable though it is apparent that service of such floors could be greatly improved if more attention were given to cleaning methods. A preliminary dampening of the surface with water before applying a solution containing an efflorescent salt is believed to overcome the greater part of the suction and greatly reduce the absorption of the solution into the pores. This practice is frequently recommended to lessen the injurious action due to crystallization of the salts in the pores.

Transporting Ready-Mixed Concrete in Open Dump Trucks*

By R. A. BURMEISTER†

SYNOPSIS

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

INTRODUCTION

The first batch of ready-mixed concrete used for a City of Milwaukee pavement project was hauled in July, 1929. The transportation unit (Fig. 1) was an open dump body equipped with several rotating paddles which traveled in vertical planes. This open dump body was used for some years but was not a success because with the standard portland cements available at that time (air-entraining cements were not then dreamed of) segregation of concrete during transit was a vexing problem. The paddles did little good, often merely cutting vertical channels through the concrete without effectively curtailing segregation. Consequently these open dump bodies were discarded in favor of the more costly but more effective, rotating drum type which really amount to traveling concrete mixers.

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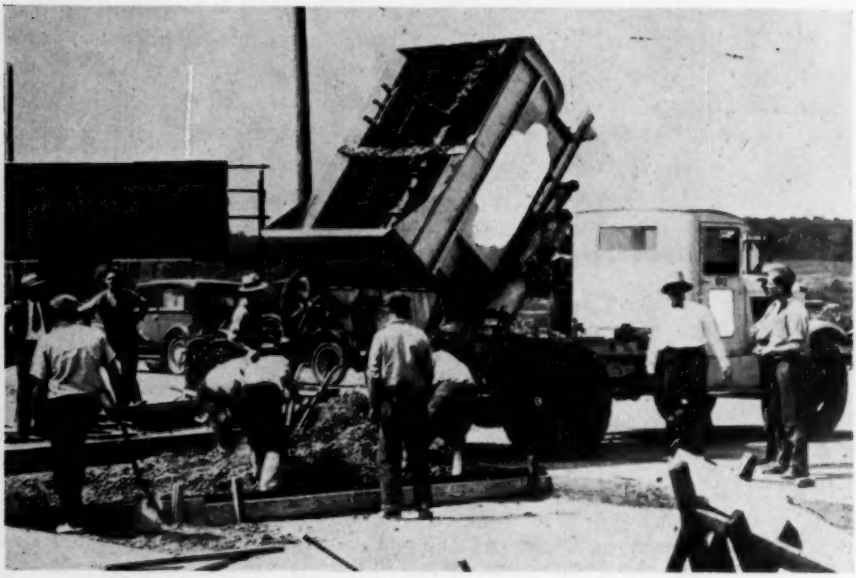


Fig. 1—The first batch of ready-mixed concrete hauled for a City of Milwaukee paving project—July 1929. Note open dump transport unit.

Recently, air-entraining cements have achieved considerable popularity and it has become apparent that concretes made with them have given us new concepts for durability, ease of placement and hauling requirements.

Milwaukee's experience with concretes having entrained air in the amount of from 3 to 6 percent by volume has shown that such concretes resist segregation to a remarkable degree. It was thought economical use of this property might be made to reduce hauling costs by use of open dump body transport units.* Transportation of entrained-air concrete in a commercial model of an open dump body truck was tried in Milwaukee in 1946 (Fig. 2) and temporary permission to use such transport units for contract work was granted August 29, 1946.

Some of the laboratory data and field tests secured in Milwaukee form the basis for this paper. These tests reported are in two groups. The first are trial runs over a route chosen for its bumpiness; these runs were made to study the practicability of the open units for hauling air-entraining concrete. A second set of tests was conducted during

Cf. information given by Alexander Foster, Jr. in "Experiences with Air-Entraining Cement in Central-Mixed Concrete", ACI JOURNAL June 1946, *Proc.* V. 42, p. 625. Foster reports unsatisfactory use of open transport units.

Also see "Improved Concreting Techniques Facilitate Grain Elevator Construction" *Engineering News-Record*, V. 139, No. 10, Sept. 4, 1947, p. 88. Concrete made with slag aggregate and air-entraining cement was successfully hauled 4 miles in open end-dump trucks, with bodies having rounded corners and double bottoms rounded at edges.

—EDITOR

Fig. 2—Open dump body transport unit used in Milwaukee tests, 1946 and 1947

work on a city contract paving job. The objective in this latter case was to ascertain uniformity of slumps and degree of segregation of concrete as normally produced by a ready-mix plant and transported in open dump body units.

TRIAL RUN OF OPEN DUMP TRUCKS

Test procedure

Aggregate samples for moisture and specific gravity tests were taken as materials entered the mixer. Batch 1 for the trial program, made in a 3 cu yd mixer at the central ready-mix plant, was discharged into

TABLE 1—TRANSIT TIME FOR TEST BATCHES

Batch No.	Time in transit, minutes	Distance traveled, miles
1	15	3.2
2	30	10.6
3	45	13.7
4	60	7.1
5	30	7.1

TABLE 2—TESTS ON CONCRETE

Batch No.	Batch 1—Hauled 15 minutes August 20, 1946				Batch 2—Hauled 30 minutes August 21, 1946			
Portion	Sampled before transit "as mixed"	Sampled after 15 minutes transit: Top 6 in. in dump body	Sampled after 15 minutes transit: First third dumped	Sampled after 15 minutes transit: Last third dumped	Sampled before transit "as mixed"	Sampled after 30 minutes transit: Top 6 in. in dump body	Sampled after 30 minutes transit: First third dumped	Sampled after 30 minutes transit: Last third dumped
Elapsed time from completion of mixing to slump test, min.	5	26	39	58	6	42	50	58
Slump test, in.	3	3	1 ¾	1½	2 ½	1 ¼	½	½
Wt. of fresh concrete, lb per cu ft	149.1	147.9	151.0	150.8	147.4	148.3	150.1	151.2
Air content, percent Gravimetric Grav. corrected	5.9 5.6	6.6 —	4.7 —	4.8 4.2	7.0 5.8	6.5 —	5.3 —	4.6 3.9
Cement content, bags per cu yd	5.99	5.94	6.06	6.05	5.93	5.96	6.04	6.09
Compressive strength, psi (28 day moist cured) Average	4020 3990 3470 3830	3850 4180 4070 4030	4290 4170 4070 4180	4080 4080 4260 4140	4130 4150 4630 4300	4280 4390 4930 4530	4660 4900 4730 4760	4150 4420 4210 4260
Workability	good	good	good	good	good	good	stiff	stiff

Ingredient	Batch 1 Cement	Batch 1 Fine agg.	Batch 1 Coarse agg. No. 1	Batch 1 Coarse agg. No. 2	Batch 1 Water added	Batch 1 Water* total	Batch 2 Cement	Batch 2 Fine agg.	Batch 2 Coarse agg. No. 1	Batch 2 Coarse agg. No. 2	Batch 2 Water added	Batch 2 Water* total
Batch weights, lb per cu yd Wet Dry	564 564	1230 1195	825 814	1225 1218	190	205	564 564	1230 1201	825 814	1225 1216	183	200
	W/C = 4.09 gal. per sack. Fine aggregate = 37.0 percent (by wt.) total aggregate. Mixing time 2½ minutes						W/C = 3.98 gal. per sack. Fine aggregate = 37.2 percent (by wt.) total aggregate. Mixing time 3 minutes					

*Includes free water on aggregates

the open transport unit and hauled some 100 yd to the test location where, by means of shovels and buckets, a sample was taken from the transport unit. This sample was representative of the concrete before the test run was made, and it is designated "as mixed". Specific weight, slump*, 6 x 12-in. cylinders and "washout" test (Fig. 3) procedures were immediately begun on the "as mixed" sample, while the truck started on its 15 minute run over 3.2 miles of fairly rough brick pavement, at 20 mph.

Following the transit run, the concrete was examined for signs of segregation, and a sample was taken from the top 6 in. The sample was obtained with shovels and buckets as before, and the same tests (except washout) were begun for this portion of the concrete.

As soon as the top 6 in. sample had been obtained, the truck dumped

*The slump test was made first, a few minutes before or after the truck started its transit run.

HAULED IN OPEN DUMP BODIES

Batch 3—Hauled 45 minutes August 22, 1946				Batch 4—Hauled 60 minutes August 23, 1946				Batch 5—Hauled 30 minutes September 10, 1946			
Sampled before transit "as mixed"	Sampled after 45 minutes transit			Sampled before transit "as mixed"	Sampled after 60 minutes transit			Sampled before transit "as mixed"	Sampled after 30 minutes transit		
	Top 6 in. in dump body	First third dumped	Last third dumped		Top 6 in. in dump body	First third dumped	Last third dumped		Top 6 in. in dump body	First third dumped	Last third dumped
8	56	63	71	6	71	77	97	9	43	51	63
3 ½	3	4	1 ½	4	1 ½	1	½	6 ½	6 ½	4	2 ½
146.8	144.5	145.4	150.1	144.7	145.1	148.0	151.6	150.2	148.9	151.7	152.9
5.2 5.3	6.7 —	6.2 —	3.1 3.1	7.7 6.9	7.4 —	5.6 —	3.3 2.9	3.2 2.1	4.1 —	2.3 —	1.5 1.7
5.87	5.77	5.81	6.00	5.79	5.80	5.91	6.06	5.97	5.91	6.02	6.07
3560 3600 3630	4200 3890 4060	3390 3440 3480	3850 3820 3490	3380 3910 3450	4210 4040 4060	3890 3910 3970	4260 4850 4800	4310 4270 4120	3930 4470 4490	4290 4540 4610	4720 4510 4430
3600	4050	3440	3720	3580	4100	3920	4640	4200	4300	4480	4550
good	good	good	good	good	good	good	good	wet	wet, segregated	good	good

Cement	Fine agg.	Coarse agg. No. 1	Coarse agg. No. 2	Water added	Water total*	Cement	Fine agg.	Coarse agg. No. 1	Coarse agg. No. 2	Water added	Water total*	Cement	Fine agg.	Coarse agg. No. 1	Coarse agg. No. 2	Water added	Water total*
564 564	1230 1204	825 814	1225 1207	210	238	564 564	1230 1204	825 814	1225 1218	207	228	564 564	1230 1198	825 816	1225 1219	235	254
W/C = 4.76 gal. per sack. Fine aggregate = 37.2 percent (by wt.) total aggregate. Mixing time 2½ minutes						W/C = 4.55 gal. per sack. Fine aggregate = 37.2 percent (by wt.) total aggregate. Mixing time 4 minutes						W/C = 5.06 gal. per sack. Fine aggregate = 37.1 percent (by wt.) total aggregate. Mixing time 4 minutes					

the first third of its load, and a sample of this material was secured for specific gravity, and slump tests and for 6 x 12-in. cylinders. The second third of the batch was then discarded, and the same tests were run on a sample from the last third of the truck load.

Batches 2, 3, 4 and 5 were sampled and tested in the same manner, except for variations in the transit run as shown in Table 1.

Compilation of test data

Table 2 summarizes the observed and computed data resulting from tests run on these five batches. The "air content–gravimetric" and cement factor determinations for the "as mixed" samples were made in accordance with the A.S.T.M. C138-44 procedure. The same procedure was followed for the other samples (top 6 in., etc.), but using the original batch weights and absolute volumes with no correction for

Fig. 3—Washout test on concrete

variation in the proportions in the concrete. Hence, the gravimetric air content and cement factors so computed are approximate.

For the "as mixed" sample, a "gravimetric corrected" air content determination was made by use of the washout test described below. For the last third of the concrete dumped, the corrected air content determination was similarly made by means of the washout test, but using original batch weights and absolute volume.

During the sampling period for these five batches, the temperature ranged from a low of 64 F to a high of 78 F. Relative humidities varied from 49 percent at 68 F to 82 percent at 70 F.*

Table 3 gives gradation and other physical data for the aggregates used in these tests. A commercial brand of air-entraining cement was used.

Washout test

The actual weight of gravel in 1 cu ft of concrete was determined by washing the concrete through a No. 4 sieve (see Fig. 3). The following is a sample computation of corrected air content based on the weight of gravel washed out.

*Complete climatological data for the five tests are available from the Institute office at the cost of reproduction.

Batch 2, "as mixed", weight per cu ft	147.44 lb
Weight of dry gravel washed out of 1 cu ft of fresh concrete (on No. 4 sieve)	62.2 lb
Weight of washout gravel retained on a ⅜-in. screen—(0.98 × 62.2)	61.1 lb
Weight of saturated surface dry gravel (1.012 × 61.1)	61.8 lb
Weight of mortar in 1 cu ft fresh concrete (147.44 − 61.8)	85.64 lb
Absolute volume of gravel—[61.1 ÷ (2.71 × 62.4)]	0.361 cu ft
Actual volume of 85.64 lb of mortar (1.000 − 0.361)	0.639 cu ft
Absolute volume of mortar $\frac{85.64}{564 + 1208 + 200}\left[\frac{564}{3.15 \times 62.4} + \frac{1201}{2.63 \times 62.4} + \frac{200}{62.4}\right]$ =	0.581 cu ft
Air content of concrete (0.639 − 0.581)	0.058 cu ft
Air content of concrete (0.058 × 1.000 × 100)	5.8 percent

In Table 4 losses in air content are listed for comparison with varying transit times.

TEST PROGRAM RESULTS

Comparison of batches

Inspection of Table 4 and Fig. 4 indicates that some air was lost during the course of transit and testing, but the air content of Batches

TABLE 3—PHYSICAL DATA ON CONCRETE AGGREGATES

FINE AGGREGATES (Sand)

Batch number	1	2	3	4	5
Source	C	C	C	C	C
Date sampled	8-20-46	8-21-46	8-22-46	8-23-46	9-10-46
Moisture, total, percent	2.9	2.4	2.2	2.2	2.7
Absorption, percent	0.8	0.6	0.6	0.4	0.6
Bulk specific gravity	2.62	2.63	2.62	2.63	2.62
Color plate	Straw	——	——	——	Clear
Silt and clay by volume, percent	2.8	——	——	——	1.0
Passing No. 200, percent	0.6	0.7	0.5	0.6	——
Sieve analysis by wt., percent passing ⅜ in.	100	100	100	100	100
No. 4	96	97	97	97	97
No. 8	78	80	80	79	78
No. 16	63	66	66	65	63
No. 30	47	49	48	48	46
No. 50	20	21	20	20	19
No. 100	2	2	2	2	2
Fineness modulus	2.94	2.95	2.87	2.89	2.95

COARSE AGGREGATES (Gravel)

Batch number	1	2	3	4	5
Source	H	H	C and H	H	H
Date sampled	8-20-46	8-21-46	8-22-46	8-23-46	9-10-46
Total moisture in No. 1, percent	1.4	1.3	1.4	1.3	1.1
Total moisture in No. 2, percent	0.6	0.7	0.8	0.6	0.6
Absorption, percent	1.4	1.2	1.0	0.9	1.0
Bulk specific gravity	2.71	2.71	2.65	2.71	2.72
Crushed particles, percent	61	64	48	58	50
Chert, percent	1.7	1.4	2.4	1.4	1.4

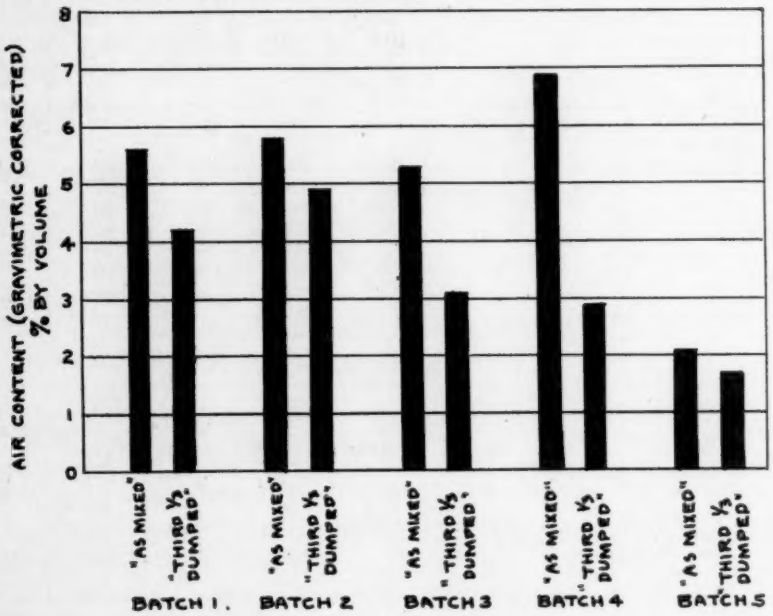
TABLE 4—GRAVIMETRIC CORRECTED AIR CONTENTS

Batch No.	Air content, percent by volume			Transit time, minutes
	"As mixed" sample	Last third dumped	Loss	
1	5.6	4.2	1.4	15
2	5.8	3.9	1.9	30
3	5.3	3.1	2.2	45
4	6.9	2.9	4.0	60
5	2.1	1.7	0.4	30

1, 2 and 3 (both before and after sampling) was within the range specified by the City of Milwaukee (3 to 6 percent by volume). These batches had initial slumps of 3, 2½ and 3½ in., with transit times of 15, 30 and 45 minutes, respectively. Compression tests made for the various samples from these three batches showed good strengths, except for the "first third dumped" Batch 3, which was slightly below normal. Hence the air content, slump and compressive strength tests indicate negligible segregation for the first three batches. This conclusion was confirmed by visual examination after transit (see Fig. 5, 6, 7, 8).

Air content loss in Batch 4 from mixing to final sampling was 4 percent. This large loss, coupled with the more than 1000 psi difference in strength

Fig. 4—Relation between air contents (gravimetric corrected) for "as mixed" and "last third dumped" samples, trial runs



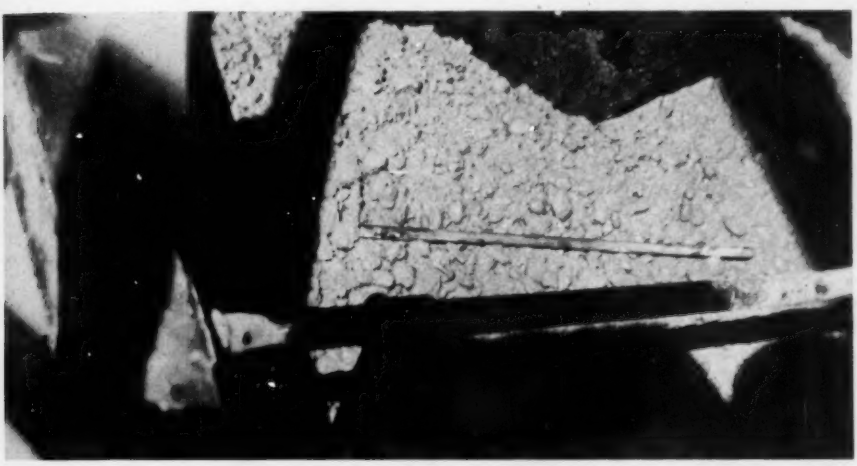
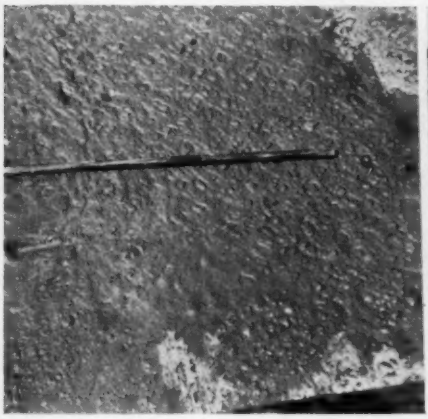


Fig. 5—Concrete has been discharged from mixer into open dump body—Batch 3. Note $\frac{5}{8}$-in. steel rod 2 ft long on surface of concrete.



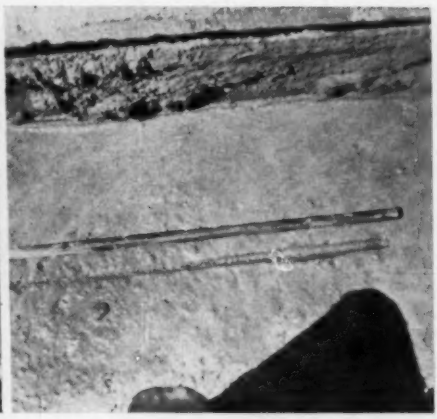


Fig. 6 (left)—Open dump body has traveled 100 yd to test location. Steel rod has been placed on concrete. The "as mixed" sample was taken at this time—Batch 3.

Fig. 7 (right)—Batch 3 after 45 minutes transit. Slight segregation visible. Note that rod has been placed on concrete twice.

values for various portions of the batch indicates that some segregation did take place. This batch had an initial slump of 4 in. and a 60 minute transit time.

Batch 5, poured several weeks later than the others, was made with cement containing less air-entraining agent, and had a lower air content with consequently higher compressive strengths. The uniformity of results from strength tests on various portions of this batch would seemingly indicate little segregation. Yet visual observation indicated

Fig. 8—Batch 3 after 45 minutes transit. Photograph taken after lapse of 71 minutes after mixing. Last third of concrete load is being dumped. Note some concrete sticking in corners.

severe segregation; the top 6 in. of the batch after transit had a 6½-in. slump and was noticeably sloppier than the bottom portion which had a slump of 2½ in. Part of the bottom portion of this batch was packed hard in the dump body and had to be picked out. The marked difference in slump due to segregation in the various portions of this batch is shown in Fig. 9.

Difficulties encountered

Getting representative samples from the dumped concrete was more difficult than getting samples from the "as mixed" and "top 6 in." portions, because there appeared to be some remixing action during the dumping.

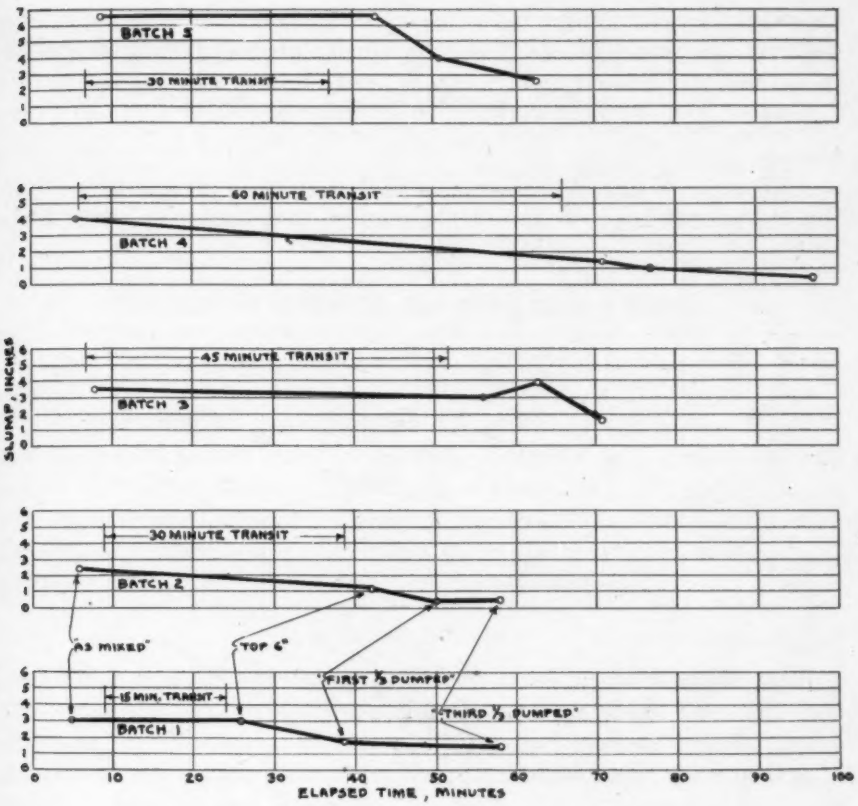


Fig. 9—Relation between elapsed time and slump, trial runs

As a means of studying the amount of mixing action incident to dumping and as a means of more accurately obtaining representative samples, some consideration was given to a method of dyeing with vivid colors various layers or portions of the load, but this idea was abandoned because of time limitations.

OPEN DUMP UNITS USED ON PAVING CONTRACT

General outline

Tests of this series were made on ready-mixed concrete being used to lay the base course for an asphalt pavement in Milwaukee, July 29, 1947. Fifty-four loads of concrete, 4½ cu yd each, were tested, totaling 243 cu yd for the day. Loads were transported in open dump body units, the same as used in trial runs previously described, except that a vibrating attachment on the body was used to alleviate the tendency for the concrete to stick in the body at the time of dumping. The

object of these field tests was to ascertain uniformity of slumps and degree of segregation of concrete as normally produced by a ready-mix plant and transported in open dump body units for a typical paving project.

Mix data

Air content of this concrete was estimated at 3 to 6 percent by volume; the 2¼ yd batches were mixed 1½ minutes in a 3 cu yd mixer. Data on batch weights are given in Table 5.

TABLE 5—MIX DATA FOR CONCRETE IN FIELD TESTS

Item	Wet batch weight, lb	Type	Remarks
Cement	1057	Air-entraining, brand 1	Nominal cement content, 5 sacks per cu yd
Fine aggregate	2892	Sand, source C	Estimated moisture, 2½ percent
Coarse aggregate No. 1	1856	Gravel, source H	Estimated moisture, ¾ percent
No. 2	2756		
Water	52.9—57.4 gal		

Test procedure

Each dump truck hauled a double batch of concrete, and slump tests were generally made at the ready-mix plant on both batches going into a load. The concrete used for these tests was taken from the truck after being discharged from the mixer. Usually when the first batch appeared too wet, the mix water for the second was reduced, or if the first batch was dry, mix water was increased on the second batch.

At the job site another slump test was made of the first two-thirds of the load as dumped. A final slump test was run on the last third of the load. Temperature during the day tests were made varied between 70 and 90 F, with relative humidity between 45 and 71 percent.*

Length of haul was about 4.9 miles with a transit time of 20 minutes. When slumps at the job site were 4 in. or less, segregation (determined by visual inspection) was negligible. For slumps over 4 in. segregation increased proportionately, becoming severe at about 6 in., and it was noted that then the bottom part of the load had a tendency to stick in the dump body. Removal of this sticking concrete was usually effected by use of the vibrator units integrally attached to the dump bodies.

Test results

These observations indicate that for practical use of an open dump unit, close control of water content must be made at the mixing plant

*Temperature data for the entire day are available from the Institute office at the cost of reproduction.

so that uniform slump of 4 in. or less can be obtained. Table 6 shows the degree of uniformity of slump obtained in these tests.

TABLE 6—UNIFORMITY OF SLUMP, FIELD TESTS

Total number of loads	54	
Number of loads for which no slump test was made on job site	4	
Number of loads for which only one slump test was made on job site	2	
Total number of loads used in following summary	48	
Number of loads having 3 in. or less slump at job site*	10 or	21 percent
Number of loads having 4 in. or less slump at job site*	36 or	75 percent
Number of loads having 5 in. or less slump at job site*	45 or	94 percent
Number of loads having 6 in. or less slump at job site*	48 or	100 percent

*These slumps are averages of first and last third of load dumped.

Thus for a normal day's run at this ready-mix plant, 75 percent of the loads were satisfactory for haulage in open dump bodies, but the remaining fourth unsatisfactory because of excessive slump. It is thought that the principal reasons for the batches having excessive slump were insufficient control and measurement of moisture contents of the aggregates by operators of the ready-mix plant.

CONCLUSIONS

These tests indicate that when air-entraining cement is used, giving 3 to 6 percent entrained air in concrete having a cement content of 5 to 6 bags per cu yd, and when not more than 45 minutes elapse from the time of mixing to the time of complete discharge from the open dump body transport unit:

1. Segregation of concrete for slumps of 4 in. and less is negligible.
2. For slumps of greater than 4 in., segregation increases proportionately and is severe at about 6 in.
3. To maintain the close control of slumps necessary for satisfactory performance of the open dump body transport units, ready-mix plants should
   a. Keep aggregates in a state of uniform gradation and moisture content.
   b. Check moisture content of aggregates at least daily and preferably several times a day and adjust batch quantities accordingly.
   c. Make slump tests during the course of each pour at ready-mix plant and at job site.

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions*

By T. M. KELLY†, L. SCHUMAN‡

and F. B. HORNIBROOK§

SYNOPSIS

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

INTRODUCTION

Stanton[1] in 1940 demonstrated that excessive expansion of concrete may occur through chemical reactions between cements of relatively high alkali content and certain mineral ingredients present in some

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aggregates. Stanton contended that the expansion thus produced was sufficient in some cases to cause disintegration of the concrete and ascribed a number of structural failures in the state of California to these expansion phenomena. Inasmuch as such failures had previously defied satisfactory explanation, his findings were received with widespread interest and other laboratories began similar studies.

The present paper supplements earlier publications[2,3,4,5] describing the results of a general investigation of alkali-aggregate reaction which was initiated at the National Bureau of Standards in 1941. In this investigation, magnitudes and rates of expansion resulting from various cement-aggregate combinations with and without certain additions were determined for mortar specimens kept in damp storage at either 70 F or 100 F. The expansion data were obtained over a period of 4 years.

Alkali type and content of the cements, and type of aggregate were introduced at the outset as test variables. Additional variables resulted from the separation of each aggregate into several size fractions, the variation in quantity of each fraction incorporated in the specimens, the addition of known percentages of various compounds (chiefly of sodium and potassium), and storage temperature.

DESCRIPTION OF MATERIALS

Four different cements, chosen to provide specific alkali contents, were employed in the investigation. The cements selected fell into the following categories: low in total alkali; high in K_2O, low in Na_2O; high in Na_2O, low in K_2O; and high in total alkali with Na_2O and K_2O content approximately equal. Throughout this paper the cements will be identified as L, K, N and T cements, respectively. Chemical analyses of the cements are presented in Table 1.

Aggregates were chosen to include materials that had been named in the literature as being reactive with alkalies, materials believed to be unreactive, and those whose alkali reactivity was unknown. The identification of the aggregates is given in Table 2.

TEST METHODS

Type and preparation of test specimens

The test specimens were in the form of 1 x 1 x 10-in. bars containing brass gage points of dimensions permitting the use of a standard autoclave comparator for measuring expansions. The bars were proportioned one part cement to two parts aggregate by weight. The aggregate in each case consisted of the selected percentage of aggregate under investigation, ranging up to 100 percent of total aggregate, and the

TABLE 1—CHEMICAL ANALYSIS AND CALCULATED COMPOUND COMPOSITION OF CEMENTS USED IN THE INVESTIGATION

Identification of cement	L (low alkali)	K (high in K_2O)	N (high in Na_2O)	T (high in total alkalies)
Chemical analysis, percent				
SiO_2	21.9	22.1	21.6	21.3
Al_2O_3	5.6	5.2	4.9	5.2
Fe_2O_3	2.4	3.4	4.3	2.6
CaO	65.9	62.3	63.7	64.2
MgO	1.4	2.9	1.7	3.1
SO_3	1.6	1.7	1.7	1.9
Ignition loss	1.3	1.4	1.2	1.3
Na_2O	0.02	0.04	1.1	0.49
K_2O	0.14	1.2	0.02	0.39
Insoluble	0.1	0.1	0.1	0.1
Free CaO	0.8	0.3	0.5	0.3
Compound composition,* percent				
C_3S	53	40	49	54
C_2S	23	33	25	20
C_3A	11	8	6	9
C_4AF	7	10	13	8
$CaSO_4$	3	3	3	3
Alkali content, percent				
Total as Na_2O	0.11	0.83	1.11	0.75
Na_2O as equivalent $NaOH$	0.03	0.05	1.42	0.63
K_2O as equivalent KOH	0.17	1.43	0.02	0.47

*Determined in accordance with the method described by R. H. Bogue, *Industrial and Engineering Chemistry, Analytical Edition*, V. 1, No. 4, 1929, p. 192.

remainder, standard 20-30 Ottawa sand. The aggregates under investigation were crushed and screened to obtain the desired size fractions. The consistencies of the mortars varied in general between a flow of 90 and 110 percent as measured by the 10-in. flow table. The mortars were mixed and the bars fabricated at a temperature of 70 F. Additions and substitutions were incorporated in the mortars in solution form as part of the mixing water, or were thoroughly dry-mixed with the cement if not readily soluble. Two bars were prepared for each particular combination of materials and test variables.

Storage of specimens

Bars for 70 F storage remained in the molds in damp storage at 70 F for the first 24 hours. They were then removed from the molds and their lengths measured. They were next placed with their lengths vertical in covered tin cans, and the cans were placed immediately in a room maintained at 70 F with relative humidity 90 to 100 percent. The surface of the bars stored under these conditions was always moist in appearance indicating that the relative humidity in these cans remained at approximately 100 percent throughout the tests.

TABLE 2—SOURCE AND PETROGRAPHIC ANALYSIS OF THE AGGREGATES STUDIED AND THEIR EFFECT ON MORTAR BAR EXPANSION

Description and source	Texture[a]	Mineral constituents[b] (approximate percent in decreasing order of abundance)		Cement	Gave excessive[c] expansion of mortar bars	
		Major (more than 3 percent)	Minor (less than 3 percent)		70 F	100 F
Opal, Virgin Valley, Nevada	amorphous	opal (relic diatom structure)	—	N,K	yes	yes
Opaline chert, Paso Robles, Calif.	amorphous	opal 90%, chalcedony 10%	trace of iron oxide	N,K	yes	—
Opaline chert, San Luis Obispo, Calif.	amorphous	opal 90%, chalcedony 10%	trace of iron oxide	N,K	yes	—
Chalcedony, Serra do Mar, Brazil	fine	fibrous quartz	—	K	yes	no
Silica brick	—	tridymite 75%, quartz 25%	—	K	yes	yes
Silica gel (dried at 110°C)	—	$SiO_2.nH_2O$	—	L	no	—
Quartz, (pale yellow), Potomac River, Md.-Va.	coarse-fine	quartz	trace of limo.	K	no	—
Chert, Tishomingo, Miss.	very fine, variable	microq., chalc.	limo.	K	no	no
Chert (Kentucky Dam area), Tenn.	very fine variable, oolitic	microq., quartz	cal., limo.	K	—	no
Chert (Novaculite) Camden, Tenn.	very fine, even	microq. (rhombic holes)	quartz	K	—	no
Chert Silver Hill, Md.	very fine-fine	microq., quartz	chalc. (rhombic holes)	N,K	no	no
Quartzite (gravel), Irvin, Wash.	—	quartz, ser., bio., cal.	musc., pc., trace titanite and leucoxene	K	no	—
Albite, Amelia Courthouse, Va.	—	albite feldspar	trace of musc., quartz	K	no	no
Microcline feldspar, Bathurst, Ont.	single twinned crystal	perthite (microcline 90%, ab. 10%)	quartz (less than ½%)	K	no	no
Muscovite	—	muscovite	—	K	—	no
Hornblende, Ontario	—	hornblende	traces of feldspar, quartz	K	no	—
Kaolin, Georgia	—	Kaolin	traces of quartz, musc.	K	no	—
Dolomite, Norwood, N. Y.	fine	dolomite 95%, quartz 5%	carbonaceous matter	K	no	—
Limestone (nodular) Bethany Falls, Mo.	very fine	calcite 98%	quartz, kaolin, mica	K	no	—
Limestone, St. Louis Co., Mo.	fine-medium	calcite, microq.	kaolin, quartz, trace py.	K	no	—
Limestone, siliceous dolomite, Paso Robles, Calif.	very fine	dol. and cal., opal 10%, chalc. 10%	—	N,K,	yes	yes
Obsidian, Lake Co., Ore. (15% of aggregate)	glassy	vol. glass 99%	feldspar laths	K	no	no
Pitchstone, New Mexico	glassy-fine	vol.glass 70%, feldspar, quartz 5%	—	K	yes	yes
Pumice	glassy-fine	vol.glass 70-90%, feldspar, quartz	—	K	no	—
Rhyolite, White Pine Co., Nevada (15% of aggregate)	—	vol.glass 50%, or. 30%, qtz. 15%	albite 3%, bio. 2%	K	no	no
Bottle glass (30-80) (15% of aggregate)	—	—	—	K	—	no
Pyrex glass (30-80) (15% of aggregate)	—	—	—	K	—	no
Slag	—	—	—	K	—	no
Granite, Bill Williams gravel, Parker Dam, Ariz.	medium	or., microline, quartz, perth., ab.	bio., ser.	K	no	—
Quartz syenite, Lyon Mt., N. Y.	medium	microline 45%, perthite 20%, ab. 20%, quartz 10%	chl., cal., mt., augite, titanite	N	no	—
Augite syenite, Lyon Mt., N. Y.	medium	or. and perthite 65%, augite 25%, ab. 5%, mt. 5%	quartz, titanite	N	no	—
Granodiorite, Spokane gravel, Irvin, Wash.	medium	pc. 50%, quartz 25%, or. 10%, bio. 5%, hb. 3%	titanite, zoisite, chl., ser. 2%, trace mt., apatite, cal.	K	no	—
Tracyhyte, Cowlitz River gravel, Wash.	fine, porphyritic	oliogoclase, hb., vol. glass	magnetite	K,N	no	—
Felsites (trachytes—andesites) Kutras gravel, Shasta Dam, Calif.	—	andesine, chl., ser.	quartz, mt., titanite, apatite	K	no	—
Felsites (rhyolites, trachytes, andesites) Bill Williams gravel, Parker Dam, Ariz.	—	feldspar, quartz, bio., vol. glass	ser., epidote, mt., chl., hema.	N,K	yes	yes
Andesite porphyry(red), Bill Williams gravel, Parker Dam, Ariz.	fine, medium porphyritic	lab., vol. glass 25%, bio., augite	magnetite	N	yes	—
Andesite, Cowlitz River gravel, Wash.	—	lab., hyperthene, vol. glass, uralite, augite	epidote, mt.	K	yes	yes

Basalt, Bill Williams gravel, Parker Dam, Ariz.	fine, porphyritic	lab. olivine (¾ altered to iddingsite), augite	mt., zeolite	K	yes	—
Basalt, Kutras gravel, Shasta Dam, Calif.	fine-medium porphyritic	lab., augite, zoisite	mt., chl., ser., microq.	N	no	—
Basalt, Lyon Mt., N. Y.	medium	pc. laths (2/3 altered to ser.), bio., augite, chl.	antigorite (after olivine), mt., apatite, cal., limo.	N	no	—
Basalt, Vantage, Wash.	very fine-fine and very coarse	black glassy matrix, 50%, pc., augite, olivine (altered to iddingsite)	—	K	no	no
Volcanic breccia, Cowlitz River gravel, Wash.	very fine-fine, very coarse	(large vol. rock fragments), pc., epidote, act., chl., microq.	mt., traces ser., cal.	N,K	no	—
Volcanic breccia, Kutras gravel, Shasta Dam, Calif.	fine and medium	(small vol. rock fragments); feldspar, vol. glass, quartz, hema., ser.	cal., chl.	N,K	no	—
Greenstone, Kutras gravel, Shasta Dam, Calif.	fine and medium	ac., quartz, epidote, feldspar, chl., ser.	—	N,K	no	—
Serpentine, Kutras gravel, Shasta Dam, Calif.	fine	antigorite (intersecting lamellas)	hb., and/or olivine, py.,titanite	K	no	—
Gneisses and schists, Bill Williams gravel, Parker Dam, Ariz.	—	hb., chl., quartz, feldspar	titanite, mt.	K	no	—
Phyllite, Buck Dam, Va.	very fine	quartz,microq. and chalc.,ser.cal.	micas, limo., trace chl.	K	yes[d]	yes[d]
Haydite	—	expanded shale	—	K	—	no
SANDS						
Cowlitz River, Wash.		andesites, feldspars, sandstones	traces secondary minerals	K	yes	yes
Elgin, Ill.		dolomite 90%, chert, granite	—	K	no	no
Potomac River, Md.-Va.		quartzites, sandstones, qtz., cht.	granite, schists, felsite	K	no	—
Saticoy, (Santa Clara River), Calif.		granites, gneisses, quartz, quartzite, cal., feldspar, sandstones, shaly and opaline sandstones	mica, opaline chert, amphiboles, chl., limo., mt., zircon	K	yes	yes
Standard Ottawa, Ottawa, Ill.		quartz	—	N,K	no	no
Hawkeye, Spokane, Wash.		quartzite, argillites, limestones, quartz, feldspar	—	K	no	yes
Mead, Mead, Wash.		same plus biotite	—	K	no	—
Florena Switch, (Blue River) Marshall Co., Kan.		quartz, feldspar, sandstones, quartzites	cherts 1%, "sandy limes" 1½%[e]	K	no	no
Florena ("sandy lime" fraction[c])		—	—	K	no	—
St. Francis (Republican River), Kans.		feldspar, quartz, sandstones, quartzites, felsites 4%	cherts 1%, opal 1%, "sandy limes" 2%[e]	K	no	yes[d]
St. Francis ("sandy lime" fraction[c])		—	—	K	yes	—
Schuyler (Platte River), Neb.		feldspar, quartz, sandstone, quartzite	felsites 1%, cherts 1%, "sandy limes" ½%	K	no	no
Kimball, Nebraska		quartz, feldspar, mica, chl., hb.	cal, felsites, iron oxides, opal	K	yes	yes
Kimball (acid washed)		—	—	K	yes	—
Kimball (calcite fraction)		—	—	K	yes	—

[a]Grain size terminology designates the following approximations: very fine-grained—less than 0.02 mm; fine-grained—0.02 to 0.2 mm; medium-grained—0.2 to 2.0 mm; coarse grained—2.0 to 10.0 mm; very coarse grained—over 10.0 mm

[b]Abbreviations used:

ab.—albite
ac.—actinolite
bio.—biotite
cal.—calcite
carb.—carbonaceous matter
chalc.—chalcedony
chl.—chlorite
dol.—dolomite
hb.—hornblende
hema.—hematite
lab.—labradorite
limo.—limonite
microq.—microcrystalline quartz
musc.—muscovite
mt.—magnetite
or.—orthoclase feldspar
pc.—plagioclase feldspar
py.—pyrite
ser.—sericite
vol.—volcanic

[c]The "calcite" fraction in Kimball sand, the "sandy-limes" in St. Francis and Florena sands are both a rather soft porous material composed of sand grains cemented with soft, calcareous matter ($CaCO_3$) and contain varying amounts of opal. (See Table 3).

[d]Excessive expansion resulted only from combinations containing additions of *KOH*.

[e]The fact that an aggregate gave no specimen expansion in the tests with high-alkali cements does not preclude the possibility of expansions resulting with other percentages or with size fractions not tested or with other storage temperatures. For example, in the case of pyrex glass the lack of expansion reported (as compared to results obtained elsewhere) might be due to differences in grading, composition or quantity used.

Bars for 100 F storage remained in the molds in damp storage at 70 F for 24 hours and then were placed in tin cans containing a small receptacle filled with water. The cans containing the bars and equipped with close-fitting covers were then stored at 100 F.

Except for their removal at periodic intervals for visual examination and length measurement, or addition of water in the case of bars stored at 100 F, the specimens remained under these respective conditions of storage during the entire period of test.

Examination and measurement of specimens

Bars were removed from storage for length measurement and visual examination at weekly intervals for the first month, at monthly intervals for the following 5 months, quarterly up to an age of one year and semi-annually thereafter. Prior to measurement the 100 F bars were taken from the can and stored for several hours in a 70 F moist cabinet. Bars showing large expansion at early ages or developing other signs of incipient deterioration were examined and measured more frequently. Visual examination consisted of close scrutiny to detect cracking, warping, discoloration or formation of visible reaction products. The progress of such defects and the change in character of reaction products were noted.

Measurements of the standard invar steel bar used to obtain the zero reading of the comparator were made at various times during the investigation. The total length change of the bar due to wear was 0.0017 in. or the equivalent of an expansion of 0.017 percent in a 10-in. specimen. Since the change to which the bar was subjected during the 4-year test period was not uniform and the rate of length change was uncertain, no attempt was made to correct the expansion measurements obtained on mortar bar specimens. It is believed that the effect on measurements made at early ages would be very small. The maximum possible discrepancy would be 0.017 percent.

RESULTS AND DISCUSSION

General

Table 1 contains the chemical analyses, the calculated compound compositions, and the alkali contents of the cements used.

Table 2 identifies all aggregates tested as to source, texture and mineral constituents, and indicates those that showed excessive specimen expansion* in at least one combination.

Specific expansion data are given in the graphs (Fig. 1 through 9). Unless otherwise noted, each point represents the average expansion of two specimens. When the average expansion exceeded 0.050 percent,

*The term "excessive expansion" as used in this paper indicates an expansion equal to or exceeding 0.050 percent.

individual expansions of a set of two specimens usually varied less than 10 percent from the set average. Occasionally, differences as high as 20 percent occurred. For lower percentage expansions, differences from the average frequently approached 30 percent. In many cases, individual expansions that showed large deviation from their average at early ages when expansions were small showed insignificant deviation at later ages when expansions were excessive.

It is recognized that the extremely large expansions appearing in the graphs are measurements made subsequent to the appearance of cracks and do not serve as a true measure of the reactive potential of an aggregate-alkali combination.

Throughout the period of test, records were kept of visible indications of chemical action and disintegration. With specimens that showed excessive expansion at any time during test the sequence of appearance of such indications was usually as follows:

1. At a very early age, often within a few days, viscous drops of gel appeared on the surface of the bars although measurement showed no expansion.

2. Gel continued to form, the quantity varying with different aggregates, and finally hardened, leaving a white deposit on the bars; the bars still showed no expansion.

3. With the first evidence of excessive expansion, that in some cases occurred at an early age and in other after 1 or 2 years, fine cracks appeared in the specimens.

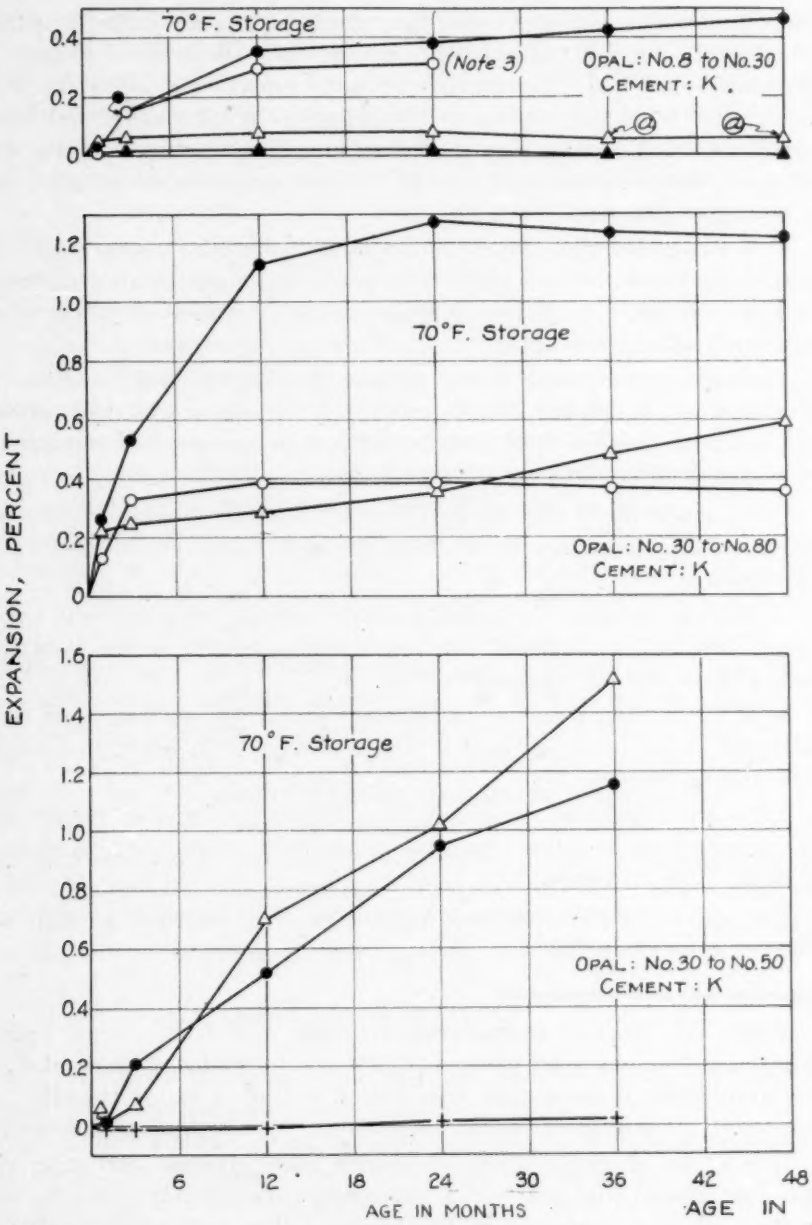
4. As the expansion increased, bars frequently became warped and the cracks became more numerous, sometimes culminating in disintegration of the specimen.

The last condition frequently occurred with specimens exhibiting delayed excessive expansion.

Expansion with certain aggregates

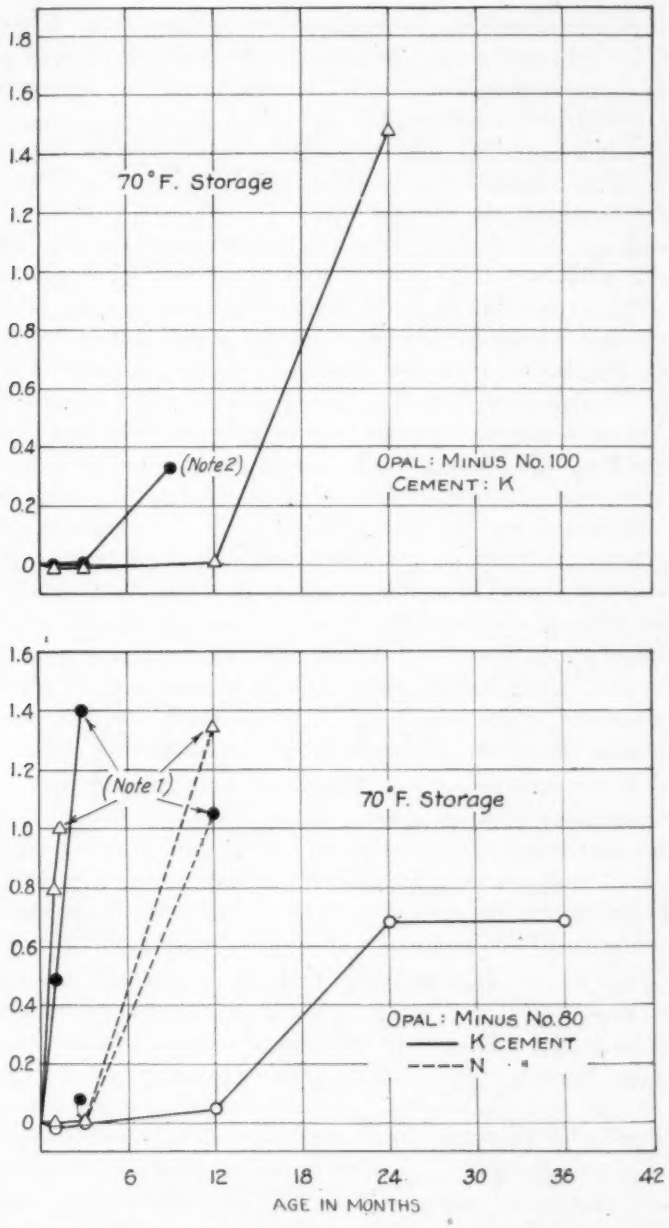
Opal—The reactive characteristics of the opal from Virgin Valley, Nevada as evaluated by expansion data from specimens cured at 70 F are graphically illustrated in Fig. 1 and 2. Fig. 1 shows the effect of variation in quantity of opal in combination with high alkali cements.

It may be observed from the graphs that, given a particular size fraction of opal, the amount of expansion varies widely with the percentage of opal contained in the mortar. The percentage producing greatest expansion is relatively small, generally between 2½ and 5 percent. In No. 8 to No. 30 size fraction, for example, the expansion with 2½ percent of opal is very large, with 5 percent, much smaller, and with 10 percent, negligible.



+— 0.5% opal; o— 1% opal; ●— 2.5% opal; △— 5% opal; ▲— 10% opal
Note 1: Subsequent expansion too great for measurement in comparator.
Note 2: Bars cracked and broken subsequent to this measurement.
Note 3: Readings discontinued.
@ point represents the expansion of only one bar.

Fig. 1—Effect of quantity of opal on mortar bar expansion



+— 0.5% opal; o— 1% opal; ●— 2.5% opal; Δ— 5% opal; ▲— 10% opal
Note 1: Subsequent expansion too great for measurement in comparator.
Note 2: Bars cracked and broken subsequent to this measurement.
Note 3: Readings discontinued.
@ point represents the expansion of only one bar.

Fig. 1—Continued

Fig. 2 illustrates the effect of opal gradation on expansion. Expansions are shown for bars containing the various size fractions of opal present in amounts equivalent to 2½ percent by weight of total aggregate, and two types of high-alkali cement.

The figure indicates that the finer sizes* of opal produce earlier and, ultimately, more disruptive expansion. The behavior of the minus No. 100 size, however, was an exception.

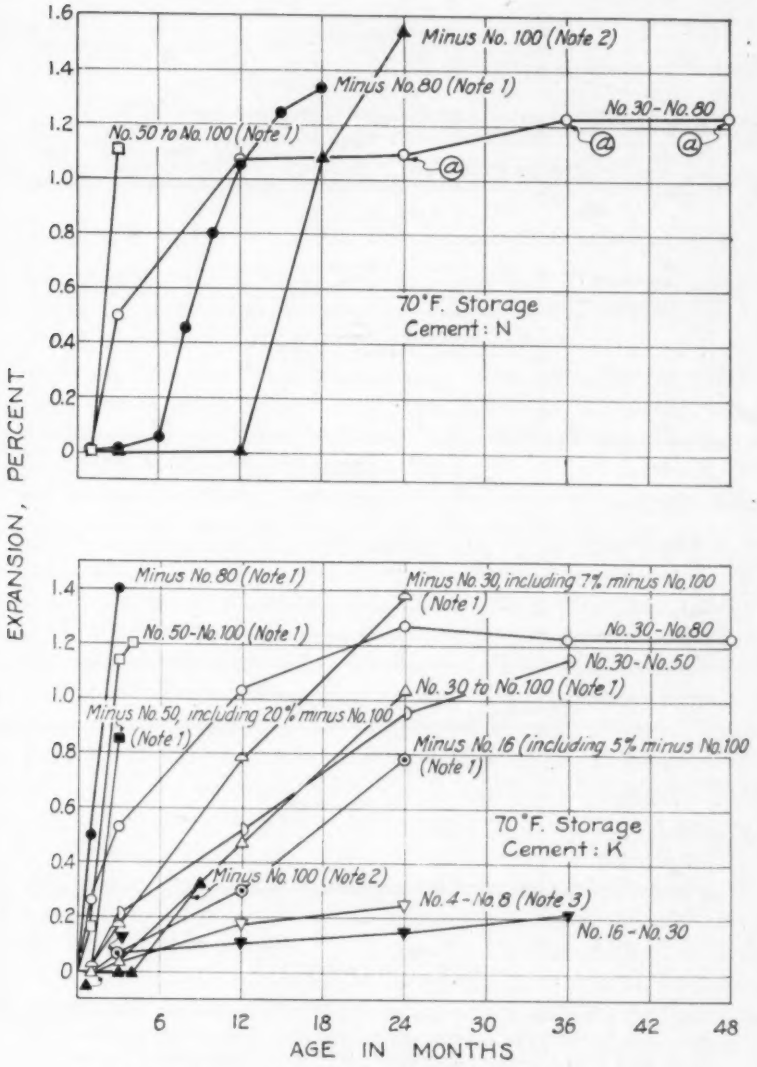
Bars made up of N cement with 2½ percent of minus No. 100 size opal (Fig. 2) and bars of K cement with 5 percent of minus No. 100 size opal (Fig. 1) showed negligible expansion up to an age of 1 year, after which they expanded steadily, finally disintegrating at the age of 2 years. Similar behavior was observed with 2½ percent of this size fraction in combination with K cement, except that expansion was negligible up to 4 months age and at 9 months the bars had expanded and disintegrated. The minus No. 80 size in combination with N cement also produced relatively low expansion up to an age of 6 months after which the expansion was rapid and considerable. In combination with K cement, however, this size produced immediate and rapid expansion and at the age of 3 months the bars had cracked and disintegrated.

Only one comparable example of such excessive delayed expansion has been found in the literature. Blanks and Meissner[7] found that with the 2 percent of $CaCl_2$ added to low-alkali cement and 19 percent of siliceous dolomitic limestone, 2 x 2 x 10-in. specimens showed little expansion up to 10 months, after which they expanded rapidly.

No. 30 to No. 80 and minus No. 100 opal in combination with L cement showed no expansion up to an age of 4 years. This is further confirmation of the belief that reactive aggregates do not produce harmful expansion, as long as the alkali content of the mortar is kept below a certain minimum. The effect of adding alkalies to low-alkali cement opal combinations is discussed later in the paper under the heading "additions."

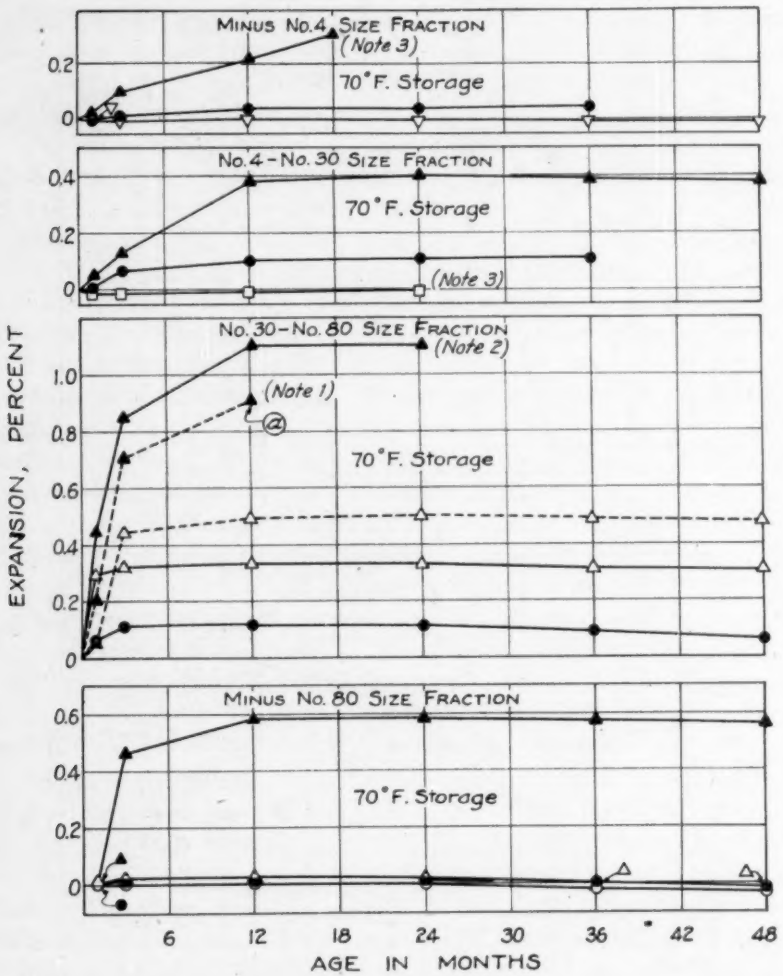
Cherts—Expansions obtained with high-alkali cements and opaline cherts from Paso Robles and San Luis Obispo, California, were excessive. The expansions approximated in magnitude those obtained with the pure opal from Nevada. The opaline cherts were composed of 90 percent opal and 10 percent chalcedony. Descriptions of cherts from other sources are given in Table 2. None of these caused excessive expansion. The data for the cherts are not directly comparable, inasmuch as the opaline chert specimens were stored at 70 F and most of the specimens containing cherts from other sources were stored at 100 F. However, the results indicate that the cherts tested (when comprising 15 percent of the aggregate) are not reactive with alkalies unless they contain a reactive substance such as opal.

*"Finer sizes," for the purpose of this discussion, include combinations of sizes all smaller than No. 30.


EXPANSION, PERCENT
Minus No. 100 (Note 2)
Minus No. 80 (Note 1)
No. 30 - No. 80
No. 50 to No. 100 (Note 1)
@
70°F. Storage
Cement: N
Minus No. 80 (Note 1)
Minus No. 30, including 7% minus No. 100 (Note 1)
No. 50 - No. 100 (Note 1)
No. 30 - No. 80
No. 30 - No. 50
No. 30 to No. 100 (Note 1)
Minus No. 50, including 20% minus No. 100 (Note 1)
Minus No. 16 (including 5% minus No. 100 (Note 1)
70°F. Storage
Cement: K
Minus No. 100 (Note 2)
No. 4 - No. 8 (Note 3)
No. 16 - No. 30
AGE IN MONTHS

Note 1: Subsequent expansion too great for measurement in comparator.
Note 2: Bars cracked and broken subsequent to this measurement.
Note 3: Readings discontinued.
@ point represents the expansion of only one bar.

Fig. 2—Effect of opal gradation on mortar bar expansion (opal content, 2½ percent)



□— 1% limestone, K-cement; ○— 2% limestone, K-cement;
●— 5% limestone, K-cement; △— 10% limestone, K-cement;
△-- 10% limestone, N-cement; ▲— 20% limestone, K-cement;
▲-- 20% limestone, N-cement; ▽— 100% limestone, K-cement;

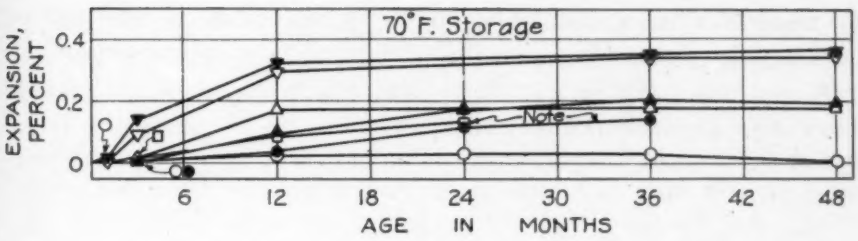
Note 1: Subsequent expansion too great for measurement in comparator.

Note 2: Bars cracked and broken subsequent to this measurement.

Note 3: Readings discontinued.

@ point represents the expansion of only one bar.

Fig. 3—Effect of quantity of siliceous dolomitic limestone on mortar bar expansion



O— 5% Pitchstone, No.30 to No.80, K-cement; ●— 15% Pitchstone, No. 30 to No.80, K-cement; △— 100% Bill William's andesite (red), minus No.4, N-cement; ▲— 100% Cowlitz River andesite, minus No.4, K-cement; ▽— 100% Bill William's felsite, minus No.4, N-cement; ▼— 100% Bill William's felsite, minus No.4, K-cement; □— 30% Bill William's felsite, No.30 to No.80, K-cement.
Note: readings discontinued.

Fig. 4—Bar expansion for various aggregates containing volcanic glasses

Siliceous dolomitic limestone—Expansion data for bars containing high alkali cements and a siliceous dolomitic limestone from Paso Robles, California are shown in Fig. 3. The limestone contained 10 percent of opal, the reactive constituent. It is apparent from the figures that expansions vary according to the amount and size fraction of the limestone in the specimens. Available data show that 20 percent of the siliceous limestone by weight of total aggregate produced greatest expansion for all size fractions. Bars composed of K cement and aggregate consisting of 100 percent of siliceous limestone (Fig. 3, minus No. 4 size) developed no expansion throughout the 4-year period of test. However, a reaction was indicated since numerous drops of gel were exuded from the bars within a period of 1 week. These hardened and dried within a month. At 1½ months additional drops appeared. No further exudations were observed. Specimens whose aggregate component was 100 percent of various types of limestone containing no opal also had developed negligible expansion up to an age of 4 years, but at no time gave evidence of gel exudation.

Aggregates containing glasses—Several aggregates containing volcanic glasses were subjected to study. These were obsidian, pitchstone, pumice and rhyolite; trachyte and andesite from Cowlitz River gravel; felsites and andesite porphyry from Bill Williams gravel; and volcanic breccia from Kutras gravel. Only the pitchstone from New Mexico, the red andesite porphyry and felsite from Bill Williams gravel, and the andesite from Cowlitz River gravel caused excessive expansion with high-alkali cements without additions in 70 F storage. Bar expansions

for these materials are shown in Fig. 4. Results obtained with 100 F storage differed somewhat from 70 F storage and are discussed later.

Basalt—Minus No. 4 size basalt from Bill Williams gravel, Parker Dam, Arizona, developed only slight expansion up to an age of 1 year, but at 2 years had caused bar expansion of 0.094 percent. Basalt from other sources such as Shasta Dam (Kutras gravel), Lyon Mt., New York, and Vantage, Washington caused no appreciable expansion throughout the test period (1½ to 4 years). In fact, bars containing glassy basalt from Vantage, Washington, showed increasing shrinkage up to 0.080 percent at 1½ years.

Chalcedony—The literature thus far has not considered chalcedony a likely source of concrete expansion resulting from alkaline reactivity.*

Porter[6] in his discussion of cherts states, "Both types of chert (opal and chalcedony) are common in California, but investigations clearly indicate that the excessive expansion with a high-alkali cement is due to the opaline type and never to the pure chalcedonic type."

Bean and Tregoning[5] found chalcedony to be highly reactive with concentrated alkali hydroxide solutions, but they remarked that it had not been associated with alkali-aggregate reactivity in the literature.

Parsons and Insley[4] reported that chalcedony was strongly etched and deeply pitted in *NaOH* and *KOH* at 90 C for 3 hours.

Chalcedony from Brazil consisting of fibrous quartz developed no appreciable expansion throughout a 4-year period at 70 F with K cement, when incorporated in specimens in amounts equivalent to 5 percent of total aggregate. However, 15 percent of chalcedony, No. 30–80 size, under the same storage conditions developed an expansion of 0.120 percent at 1½ years after showing negligible expansion up to an age of 1 year. The expansion continued and at 3 years had reached 0.368 percent. A few drops of gel were observed at 1 week, several more at 1 month and many more at 6 months. The bars were slick and slimy at 2 years. Similar bars in 100 F storage showed gel exudation at early ages but developed negligible expansion throughout the 3-year period of test.

Silica brick—Using K cement, specimens were prepared in which part of the aggregate was replaced with various percentages of ground silica brick containing a large amount of tridymite. All percentages caused some expansion (ranging at 1 year from 0.030 to 1.140) at both 70 F and 100 F storage temperature. The greatest occurred with 15 percent of silica brick and 100 F storage (0.400 percent at 3 months;

*In an article that appeared subsequent to the preparation of this paper, Duncan McConnell and co-workers (in "Cement-Aggregate Reaction in Concrete," ACI JOURNAL Oct. 1947, *Proc.* V. 44, p. 93) cite several examples of excessive expansion of mortar bars containing various percentages of chalcedony and stored at 100 F.

TABLE 3—OPAL AND VOLCANIC GLASS CONTENT OF KANSAS AND NEBRASKA SANDS

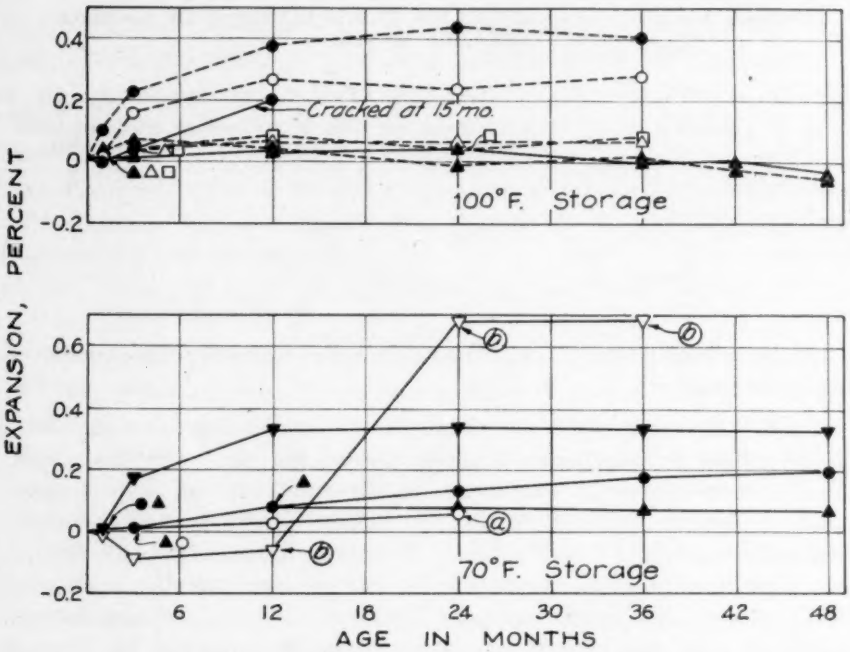
Sands	Approximate percent of opal			Percent volcanic glass in felsite
	"Sandy limes" or calcite fraction	Fine sands	Coatings	
Kimball, Neb.	0.4	0.1	0.4	0.5
St. Francis, Kans.	0.4	0.1	—	1.0
Schuyler, Neb.	0.2	0.05	—	trace
Fremont, Neb.	0.2	0.05	—	trace
Florena Switch, Kans.	—	—	—	none

1.140 percent at 1 year). Additions of alkalies increased the expansions at later ages.

Sands—In combination with high-alkali cements, the following sands gave excessive expansion: Kimball, Nebraska sand; Saticoy sand, (Santa Clara River) California; the "sandy-lime" fraction of St. Francis sand, (Republican River, Kansas); the calcite fraction of Kimball, Nebraska sand; and Cowlitz River, Washington, sand. Except for the last mentioned, opal was probably the reactive constituent in each case. Shaly and opaline sandstone were major constituents of the Saticoy sand and opal was present as a minor constituent of the St. Francis and Kimball sands. The effect of the opal in the two latter sands was greatest when the "sandy-lime" (minus No. 30) and calcite (No. 30 to 80) fractions were separated from the parent sands and incorporated in expansion bars, replacing, respectively, 25 and 20 percent of the standard sand aggregate. However, minus No. 4 size of Kimball sand as received gave appreciable expansion at 1 year, and even after it had been acid-washed gave an expansion of 0.068 percent at 1 year. (The percentages of opal and volcanic glass in Kansas and Nebraska sands are given in Table 3.) The glassy andesite in the Cowlitz sand is suspected as the reactive constituent. Parsons and Insley[4] report an etching of glassy andesite in solutions of *NaOH* and *KOH*.

Greatest expansion with Saticoy sand resulted when the minus No. 4 size constituted 100 percent of the aggregate and the expansive effect was increased by 100 F storage and additions of alkali. Specimens containing 100 percent of Cowlitz River sand, minus No. 4, showed expansion of 0.066 percent at 2 years age in 70 F storage. Storage at 100 F plus 1 percent *KOH* addition caused earlier and greater expansion (0.16 at 3 months, 0.27 at 1 year.).

Hawkeye sand, Spokane, Washington and St. Francis sand (Republican River) Kansas gave excessive expansion only in combinations to which 1 percent of *KOH* had been added. The bars were made with high K_2O cement and stored at 100 F.



○— Cowlitz, minus No.4 ; ○--- Cowlitz, minus No.4, plus 1% KOH;
●— Saticoy, minus No.4 ; ●--- Saticoy, minus No.4, plus 1% KOH;
△--St. Francis, minus No.4, 1% KOH; ▽— St. Francis, "sandy limes"; ▲— Kimball, minus No.4; ▲--- Kimball, minus No. 4, plus 1% KOH; ▼— Kimball, "calcite fraction"; □--- Hawkeye, minus No. 4, plus 1% KOH.

@ The readings discontinued.

ⓑ point represents the expansion of only one bar.

Note: Sands comprised 100% of total aggregate, except for "calcite fraction" and "sandy limes" which comprised 20% and 25%, respectively.

Fig. 5—Bar expansions for various sands with cement K

Fig. 5 shows expansions resulting with certain combinations of sands and cement.

Effect of storage temperature on specimen expansion

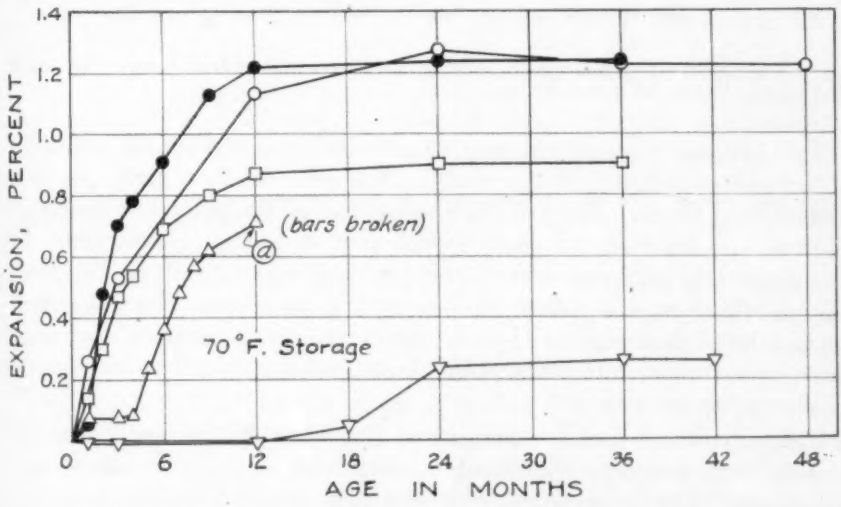
Where the data permitted, comparisons were made between 70 F and 100 F storage. The effects of storage temperature were not consistent. Expansion of high-alkali cement specimens containing opal, pitchstone, Saticoy sand or silica brick were appreciably accelerated by 100 F storage. Conversely, the higher temperature reduced from appreciable to negligible the expansion of specimens containing chalce-

dony or Cowlitz River andesite (without alkaline additions) and appreciably reduced expansion of specimens containing siliceous dolomitic limestone. Kimball sand (minus No. 4) with high K_2O cement also showed less expansion at 100 F. Buck Dam phyllite and cements high in Na_2O or K_2O did not develop excessive expansion in either 70 F or 100 F storage. With this same aggregate, K cement and KOH addition, bars in 70 F storage showed excessive expansion only after 3 years (0.069 percent). Similar bars stored at 100 F had expanded 0.190 percent at one year.

Effect of additions and substitutions on specimen expansion

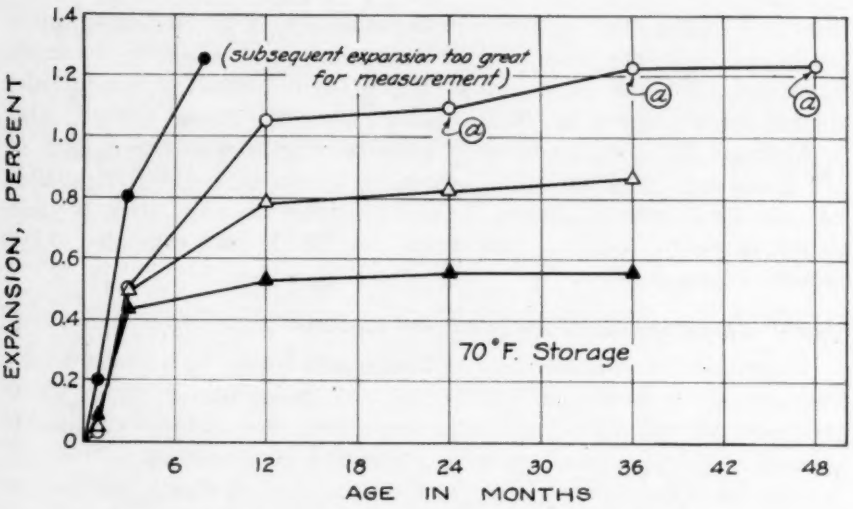
Materials other than cement and aggregate which were added to the mortars fell, generally, into either of two classifications: those which increased the alkalinity of the mortars and those added as possible correctives to reduce the expansion of reactive combinations. Normally, the materials were added to mortars consisting of one part cement and two parts aggregate. In some instances the additions were actually replacements of a certain percentage of either cement or aggregate.

Fig. 6—Additions of potassium compounds to mixtures of low-alkali cement and opal (2½ percent of No. 30 to No. 80 size)



○— K-cement, no addition; ●—L-cement, plus 1.9% KCl;
□— L-cement, plus 2.2% K_2SO_4; △— L-cement, plus 1.4% KOH;
▽— L-cement, plus 1.75% K_2CO_3

Note: The addition plus the K_2O content of L-cement gives in each case an equivalent K_2O content approximately equal to that of K-cement.
@ point represents the expansion of only one bar.



○— N-cement, no addition ; ●— L-cement, 1.4% NaOH;
△— L-cement, 2.0% NaCl ; ▲— L-cement, 2.5% Na_2SO_4
Note: The addition plus the Na_2O content of L-cement gives in each case an equivalent Na_2O content approximately equal to that of N-cement.
@ point represents the expansion of only one bar.

Fig. 7—Additions of sodium compounds to mixtures of low-alkali cement and opal (2½ percent of No. 30 to No. 80 size)

Test bars containing alkaline additions were prepared for the majority of aggregates included in the study. The effect of such additions was investigated most comprehensively using opal as the reactive aggregate. Various size fractions of opal incorporated in amounts equivalent to 2½ percent of total aggregate, three types of cement, and several compounds of sodium and potassium were used in the study. Space does not permit detailed discussion of each combination of materials, and only those of particular interest or which best illustrate the effect of alkaline additions are discussed.

Using L cement and 2½ percent of No. 30 to 80 size opal, a group of bars was prepared containing various compounds of sodium and potassium. The compounds were added in amounts which, together with the alkalies in the cement, approximated the equivalent K_2O or Na_2O content of the high-alkali cements used in the investigation. By means of the data thus obtained, some comparison was possible between added compounds and alkalies present in cements. Furthermore, the data provided an approximate measure of the relative reactivity of the various compounds of sodium and potassium. These data are

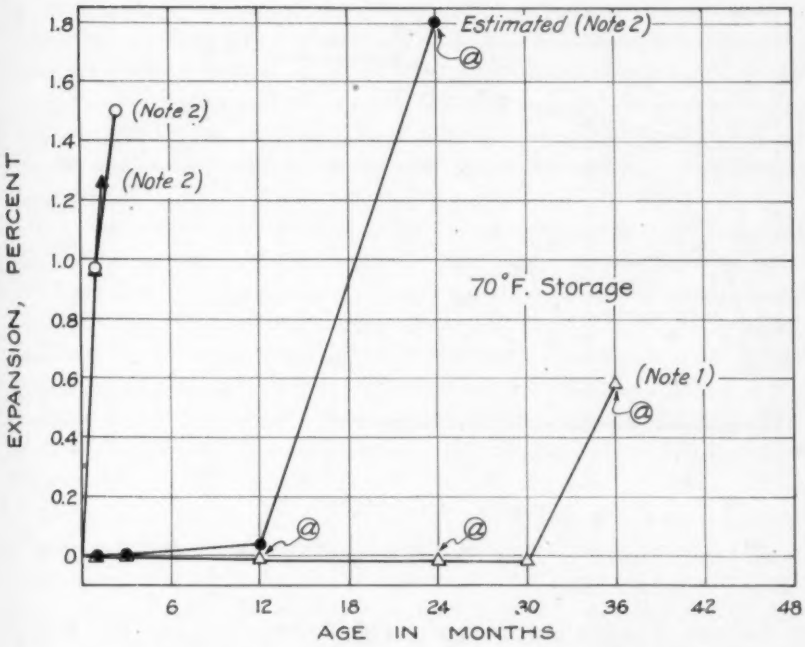


Fig. 8—Additions of potassium compounds to mixtures of low-alkali cement and opal (2½ percent of minus No. 80 size)

presented graphically in Fig. 6 and 7. With the exception of 1.4 percent of $NaOH$ and 1.9 percent of KCl, sodium and potassium additions to low alkali cement caused less expansion than the high Na_2O and high K_2O cements of equivalent alkali contents.

Fig. 8 and 9 illustrate the effect of adding different amounts of sulfates of potassium and sodium to mortars containing L cement and 2½ percent of minus number 80 size opal. The smallest percentage additions of the sulfates gave greatly delayed expansion (24 to 30 months). With increased percentages (1.1 percent of K_2SO_4 and 1.2 percent of Na_2SO_4) expansion began at 12 months. On the other hand, excessive expansion at early ages was produced by 2.2 and 2.5 percent of K_2SO_4 and Na_2SO_4, respectively. The 2.5 percent of Na_2SO_4 produced less (though still excessive) expansion at later ages than did an addition of 1.2 percent. Data at later ages were not available for the 2.2 percent addition of

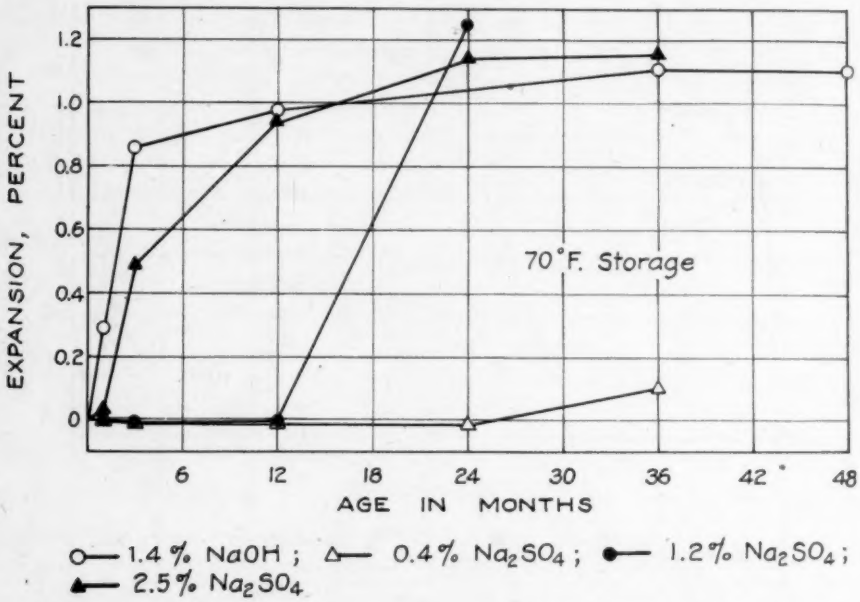


Fig. 9—Additions of sodium compounds of low-alkali cement and opal (2½ percent minus No 80 size)

K_2SO_4, since one bar was broken at 2 months and the other was too long for measurement in the comparator. The delayed character of the expansion for the lower percentage additions is perplexing. It is comparable to the behavior observed with the very fine sizes of opal in combination with high-alkali cements.

From the data available, it appears that additions of *KOH* to combinations of opal and K cement cause increased expansion at all ages. Conversely, additions of *NaOH* to opal and N cement tend to accelerate the expansion but lessen it at later ages. However, the data are too meager to warrant definite conclusions.

The addition of non-alkaline materials and compounds to opal-high-alkali mixtures was largely ineffective in reducing expansion. In 70 F storage, 3 percent of magnesium fluosilicate caused practically no changes in the amount of the expansion. Replacing 1½ percent of K cement with an equivalent weight of acetic acid accelerated the rate of expansion. Five percent of calcium hydroxide also accelerated expansion. However, the substitution of diatomaceous earth for a relatively large portion of the cement (25 percent) was very effective.

Bars in 70 F storage containing N cement and five percent of No. 30 to 50 opal with 25 percent of the cement replaced by diatomaceous earth

showed a negative expansion throughout a 3-year period of test.* Similar bars in which 25 percent of the cement was replaced with soda feldspar showed expansion of 0.480 percent at 3 months and at 9 months were too long to measure.

The expansion in 70 F storage of bars containing 2½ percent of No. 50 to 100 opal and N cement was delayed by the addition of 4 percent of dehydrated kaolin. The expansion measured 0.018 percent at 1½ years, 0.556 percent at 2 years and 1.406 percent at 3 years. Bars without the kaolin addition had expanded 1.100 percent at 3 months and were too long for the comparator at subsequent periods. Negligible expansion resulted from small percentage additions (2½ to 10 percent) of kaolin to bars containing high-alkali cement and no reactive aggregate. When larger amounts of kaolin were added (as either cement or aggregate replacement) to opal-high-Na_2O combinations, expansions were excessive at both 70 and 100 F and were greatly accelerated at the latter temperature.

Expansions caused by 20 percent of the siliceous dolomitic limestone, L cement, and 1.4 percent of *KOH* addition were excessive. However, they were smaller than those resulting with high K_2O cement and no additions, though the equivalent K_2O content was the same for each mixture. Additions of 1 percent of *KOH* to obsidian specimens made with high K_2O cement caused no expansion. Expansions of pitchstone and silica brick specimens made with K cement and stored at 100 F were increased by a 1 percent *KOH* addition.

With K cement, Cowlitz River sand, Saticoy sand, Spokane, Washington sand (Hawkeye), and St. Francis sand, all in 100 F storage, and with andesite from Bill Williams gravel (Parker Dam) in 70 F storage, the expansion of the specimens increased as a result of 1 percent of *KOH* additions. Kimball, Nebraska, sand with 100 F storage and 1 percent of *KOH* addition showed a decrease.

Buck Dam phyllite (100 percent of No. 4 size) and K cement showed only slight expansion in both 70 and 100 F storage. With the addition of 1 percent of *KOH*, bars at 70 F had expanded 0.030 percent after one year and 0.069 percent after 2 years. The expansion was further accelerated by 100 F storage. It measured 0.190 percent at 1 year and decreased to 0.095 at 3 years.

Several aggregates of igneous type combined with high-alkali cements, and certain combinations containing L cement and opal or siliceous dolomitic limestone exhibited viscous exudations at a relatively early age. However, they failed to develop excessive expansion at any time during the test period.

*Meder found that small percentage additions of diatomaceous earth (1 to 3 percent) increased the expansion.

SUMMARY

With the exception of chalcedony and silica brick, the materials that gave excessive expansion have been cited for their reactivity in the literature.

Opal and opaline cherts were most reactive with alkalies and gave greatest expansion when added in relatively small percentages. The percentage for greatest expansion varied between 2½ and 5 percent, depending upon the size fraction of opal. This conclusion is in general agreement with the work of Stanton[6]. Data for bars containing large amounts of opal (10 percent) are available for only one size fraction (No. 8 to No. 30). The expansion of these bars over a 4-year period was negligible.*

The finer size fractions of opal (graded from No. 30 to finer sizes) usually gave earliest and highest expansions. However, the minus No. 100 size, when comprising the entire opal constituent of 2.5 to 5.0 percent of the aggregate, delayed expansion for a period varying from several months to 1 year; at the end of this period of inactivity, expansion was rapid and disruptive.

Comparison of specimens containing cherts in the amount of 15 percent of the aggregate from various sources including opaline chert indicates that this rock type when present in the above percentage is not reactive with alkalies unless it contains a reactive material such as opal.

A siliceous dolomitic limestone (containing 10 percent of opal) gave excessive expansions which reached a maximum when the limestone comprised 20 percent of the total aggregate and bars were stored at 70 F. A reduction in percent of expansion resulted from 100 F storage.

Expansions resulting with aggregates containing volcanic glasses did not establish definitely the alkaline reactivity of this constituent. Not all of the aggregates containing glasses gave excessive bar expansion. Those that did were pitchstone from New Mexico, red andesite porphyry and felsite from Parker Dam, and andesite from Cowlitz River, Washington.

Chalcedony gave excessive expansion at later ages in 70 F storage when the No. 30 to 80 size comprised 15 percent of total aggregate. Identical specimens in 100 F storage failed to show appreciable expansion throughout a 3-year period of test.

Tridymite in ground silica brick caused excessive specimen expansion. Fifteen percent of silica brick at 100 F storage, the maximum amount tested, gave the greatest expansion.

Expansions observed among the sands investigated were, with one exception, traceable to the opal constituent. The exception, Cowlitz

*Blanks and Meissner[7] state that ". . . added in sufficient quantity, the reactive aggregate provides its own curative."

River sand, was apparently not reactive with high-alkali cement until specimens had aged 2 years, at which time the expansion measured 0.066 percent. At 100 F storage, the addition of 1 percent of *KOH* caused earlier and greater expansion. Glassy andesite is suspected as the reactive constituent.

Aggregate-high-alkali cement combinations were variously affected by the higher storage temperature. Expansions increased in some instances and decreased in others. The general effect of temperature variation was not great.

Additions of sodium and potassium compounds to reactive-aggregate specimens made with L cement were apparently less effective in producing expansion than equivalent alkalies present in high-alkali cements.

The effect of adding alkalies to reactive combinations made with high-alkali cement was not predictable. Normally expansions were increased by such addition, but infrequently a decrease resulted.

Diatomaceous earth in sufficient quantity as a cement replacement (25 percent) was very effective in eliminating excessive expansion. Other materials, including dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide were largely ineffective.

The development of a dependable accelerated test to detect harmful aggregate-alkali combinations is greatly to be desired. The several instances of delayed expansion encountered in this study are indications of the weakness of the mortar bar test. The greatly delayed and disruptive expansions resulting from minus No. 100 size opal and both high Na_2O and high K_2O cements are disturbing and indicate the need for further study of the effects of very fine sizes of reactive aggregates.

ACKNOWLEDGMENT

The assistance of Willard H. Parsons, formerly a member of the staff of the National Bureau of Standards, is gratefully acknowledged. Doctor Parsons made the petrographic analyses of all aggregates used in the investigation.

Acknowledgment is also made to R. A. Clevenger and N. Green, present and former members of the staff, respectively, for fabrication and measurement of many of the specimens.

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Discussion of a paper by T. M. Kelly, L. Schuman, and F. B. Hornibrook:

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions*

By BRYANT MATHER, THOMAS E. STANTON and AUTHORS

By BRYANT MATHER†

The paper under discussion is a valuable contribution to our knowledge of alkali-aggregate reactivity and the authors are to be particularly commended for having devoted considerable attention to the accurate determination of which aggregate materials were reacting with the cement. The importance of this aspect of the problem is often neglected.

The purpose of the present discussion is similar to that of the writer's previous discussion[1] of an earlier paper[2] reporting studies conducted by the National Bureau of Standards. The authors, in their discussion of expansion of bars containing chalcedony (p. 70), state that "the literature has thus far not considered chalcedony a likely source of concrete expansion resulting from alkaline reactivity." They then note that Bean and Tregoning[2] found chalcedony to be highly reactive with concentrated alkali hydroxide solutions and that Parsons and Insley[3] found that chalcedony was strongly etched and deeply pitted in sodium hydroxide and potassium hydroxide. The earlier discussion[1], by the writer, of the paper by Bean and Tregoning invited attention to the fact that the material known as chalcedony consists of fibers composed of low-temperature quartz and a continuous interstitial material having all the properties of opal. It was also pointed out that the individual particles composing the fibers of chalcedony are of sub-microscopic size and may be smaller than the wave length of light. The suggestion was advanced at that time that the reactivity of chalcedony was a function of the amount of opal contained. Alderman, Gaskin, Vivian, and Jones[4] suggest that very careful work is necessary to distinguish cryptocrystalline quartz and chalcedony and have raised the question that crypto-

*ACI JOURNAL, Sept. 1948, *Proc.* V. 45, p. 57. Disc. 45-5 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45.

†Concrete Research Engineer, Concrete Research Division, Waterways Experiment Station, Mississippi River Commission, Corps of Engineers, Clinton, Mississippi.

crystalline or microfibrous quartz itself may be reactive. The following paragraph is quoted from their report.

"In several aggregates which reacted with high alkali cement it was not possible to identify any material, such as opal, which had been found by previous workers to be reactive. These aggregates all showed 'delayed' reaction which did not appear until the mortar was about a year old. The reactive material seemed to be cryptocrystalline silica which was first thought to be chalcedony. Chalcedony, the 'chalcedonite' of Winchell (1933), has an average refractive index $n = 1.533$, which is lower than the corresponding index of quartz which is 1.547. The refractive index of the cryptocrystalline silica in these aggregates was determined and found to be that of quartz. Thus this material must be called cryptocrystalline quartz. Opal could conceivably be intergrown with the quartz but very careful microscopic examination did not disclose it."

It is believed important to emphasize that chalcedony can not be distinguished from cryptocrystalline quartz on the basis of structural or textural criteria. The distinction between these two materials must be based on determination of properties such as index of refraction, specific gravity, or water content. It is believed that index of refraction is the most convenient diagnostic property. The use of index of refraction to distinguish cryptocrystalline quartz and chalcedony has been discussed by the writer[5] and used by Bramlette[6] and others.

It might also be noted that Table 3 of the paper by McConnell, Mielenz, Holland, and Greene[7] referred to by the authors in the footnote (p. 70), in which chalcedony is listed as a "mineral" which is "deleteriously reactive with high-alkali cement" was previously published in identical form as Table 1 of the April 1946 paper by Mielenz[8]. The earlier paper, however, presented no supporting data to indicate the basis on which it had been determined that chalcedony was reactive and neither paper gave the criteria by which the materials so identified were distinguished from cryptocrystalline quartz.

Now that the generally unfavorable chemical reactivity of chalcedony is becoming better known, it is hoped that more detailed work will be conducted to establish the details of the phenomenon. Particular matters on which to the best of the writer's knowledge no data are available are variation in degree of reactivity of chalcedony as a function of its opal content and reactivity of microfibrous quartz.

It is suggested that investigators of the behavior of minerals and rocks with respect to alkali-reactivity make every effort to include petrographic data in the reports of their work so that specific physical and chemical properties of the materials studied will be a matter of record for the guidance of other workers in this highly complex and extremely important problem.

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By THOMAS E. STANTON*

The authors have presented considerable data of an interesting and valuable nature. The data, however, emphasize the frequent disparity in test results by different operators under different conditions.

It is doubtful if any phenomenon of concrete performance has developed as much contradictory test data as the excessive expansion of concrete through some incompatibility of the cement and aggregate.

The purpose of this discussion is to set forth some of the observations of other operators (primarily the writer) which differ in some respects from those of the authors as well as some in substantial agreement.

Part of this contribution consists in bringing up to date the test data presented by the writer at the January 1940 meeting of A.S.C.E.† at which time the test data covered only four to six months.

PARTICLE SIZE

The authors produce considerable evidence showing that while 2½ percent of the minus 100-mesh size of a reactive mineral (opal) induced negligible expansion up to one year with the N Cement, thereafter there was a steady expansion and, finally, disintegration at 2 years.

California experience with high soda California cements has been the reverse.

Fig. 6(b) of the 1940 A.S.C.E. California report showed negligible expansion in six months with 5 percent of the minus 80-mesh particles of the reactive opaline chert No. 28039 substituted for an equivalent percentage of sand. The expansion was still negligible at four years, whereas the expansion with the coarser sizes of the same reactive aggregate had progressively increased as shown by the following data:

*Materials and Research Engineer, Calif. Div. of Highways.
†*Proceedings*, A.S.C.E., V. 66, 1940, pp. 1781-1811.

Sieve size of particles of reactive mineral	Percent expansion of mortar bars of different size particles of siliceous magnesian limestone (Opaline chert No. 28039)	
	1:3 mix	5 percent No. 28039
	6 months	4 years
− No. 80	−0.007	0.003
+ No. 80 − No. 30	0.501	0.810
+ No. 30 − No. 10	0.312	0.552
+ No. 10 − No. 3	0.196	0.536

EFFECT OF PERCENTAGE OF REACTIVE PARTICLES

Fig. 5 of the 1940 A.S.C.E. report shows the corrective effect of high percentages of the reactive rock No. 28039 (siliceous magnesium limestone containing approximately 20 percent opal). Fig. 5 of a subsequent paper published in the ACI JOURNAL, Jan. 1942, *Proc.* V. 38, p. 216, shows similar data for a high opal content rock (approx. 100 percent opal) in which it is shown that comparatively small percentages of the opal constituent have the same apparently corrective properties.

In view of the authors' discussion of the same phenomena it is believed of interest to present the 4 year test data on the specimens in the original study, which were fabricated in 1939 and 1941. A high alkali cement (1.04 percent as Na_2O) was used in this series.

From this table it will be seen that while the lower opal content chert No. 28039 becomes adversely reactive when present in quantities greater than one percent the action does not reach a peak until 20 percent, after which the measurable effects of the reaction drop, the drop becoming very marked at 40 percent with negligible measurable reaction at 50 percent and higher substitutions.

With the approximately 100 percent opal rock the measurable effects of the reaction are excessive when present to the extent of less than one quarter of one percent, rise to a peak at 5 percent and then decline and finally completely disappear at 15 percent opal.

These data illustrating the effects of small variations in percentage of highly reactive minerals account at least in part for the difficulty in securing consistent results between different and even in repeat tests by the same operators and why repeat tests on apparently identical samples of aggregate frequently show radically different results.

THE EFFECTS OF TEMPERATURE

The authors noted that the reactive aggregate-high alkali cement combinations were variously affected by higher than normal storage temperatures; the expansions being increased in some instances and

Percent of reactive rock particles substituted for an equivalent percentage of non-reactive sand	Type of reactive rock or mineral	
	Opaline chert No. 28039, approx. 20 percent opal	Opalized wood, approx. 100 percent opal
	Percent expansion in 48 months	
0.25	0.039	0.477
0.50	0.034	0.565
0.75	0.053	0.573
1.00	.....	.....
1.25	0.127	.....
2.00	.....	1.407
2.50	0.268	.....
3.00	.....	1.962
4.00	.....	2.061
5.00	0.399	2.391
10.00	0.689	0.652
15.00	.....	0.019
20.00	1.208	.....
30.00	1.145	.....
40.00	0.115	.....
50.00	0.018	.....
60.00	−0.006	.....
70.00	−0.004	.....
80.00	−0.009	.....
90.00	−0.013	.....
100.00	−0.011	.....

decreased in others. The same phenomena has been noted by others and to some extent is checked by the California studies. The preponderance of evidence, however, indicates an accelerated result at 100 to 110 F even though the ultimate expansion may occasionally be less at the higher temperature. This conclusion is confirmed by hundreds of tests conducted in California* on over 50 brands and types of cement.

It will be noted that in every case heat accelerated the action except in the case of the high potash cements 24, 25, and 26. Other tests with all of the California commercial brands of cement (all of which are relatively low in potash) are in accord.

It is true that the authors state that "Expansion of high-alkali cement specimens containing opal, pitchstone, Saticoy sand or silica brick were appreciably accelerated by 100 F storage." At the same time they recite that the higher temperature reduced appreciably the expansion of specimens containing siliceous dolomitic limestone.

In Table 2, the source of the siliceous dolomitic limestone is given as Paso Robles, California. As this is the area from which the so-called siliceous magnesian limestone No. 28039, used in the California tests was secured, it is reasonable to assume that the two are similar. The reactive mineral in the siliceous magnesian limestone was conclusively

*A.S.T.M. *Proceedings* V. 43, p. 886, Fig. 3 and 4.

determined to be opal which in the material used in the California series was found to be present to the extent of approximately 20 percent. This is the reactive rock used in the series for which the results were tabulated in Fig. 3 of the A.S.T.M. paper wherein it is shown that temperature accelerated the action except with high potash cements.

Fig. 5 of the same A.S.T.M. paper illustrates similar results with a number of sands from different sections of the state. The same results have been observed with reactive California andesites.

DIATOMACEOUS EARTH AS A CORRECTIVE

The authors point out that diatomaceous earth in sufficient quantity (25 percent by weight) as a cement replacement was found to be very effective in eliminating excessive expansion.

In the original California studies it was found that the normal usable percentage of Celite (3 percent by weight) was ineffective.

Because Celite is so light (less than 10 lb per cu ft for the better grade) the bulking effect and water demand is so great that material loss in strength can be expected if much more than 3 percent by weight is used.

While the authors do not intimate that they consider a 25 percent by weight of cement substitution to be practicable, the method of presentation might be interpreted as carrying the inference that this high percentage of diatomaceous earth may be a practicable, and hence suitable, substitute for other relatively high specific gravity siliceous pozzolans.

Furthermore, the bulking effect on the entire mortar batch, where the proportions are 0.75 cement: 0.25 diatomaceous earth: 2.00 sand, by weight, is such as to materially reduce the percentage of cement in the combined mix and, therefore, the percent of alkali in the combination, to a point where it is possible that no measurable adverse reaction would be observed without the diatomaceous earth.

A series of tests was therefore indicated, designed to determine how much, if any, the observed reduction in expansion could be attributed to any corrective properties of the diatomaceous earth and how much to the reduction in alkali resulting from the reduction in cement.

A series of tests was undertaken recently to secure data on this phase. As California uses 1:2.25 by weight proportions, the specimens were fabricated on that basis.

The cement factor in a 1:2.25 mortar equals approximately 11 sacks per cu yd, whereas with a 25 percent substitution of Celite by weight of cement and the sand weight maintained constant, the cement factor becomes approximately 6.1 sacks per cu yd. Therefore, some specimens were fabricated in which the cement content was reduced to the equivalent of that in the mortar of the set of specimens to which the Celite had been added, but with no Celite, the difference in volume being made up by an increase in the sand content.

The following table shows the test results up to 28 days, all curing being in sealed containers at approximately 110 F.

Mix No.	Composition of mix (All proportions by wt)	Percent expansion, Age in days				28-day compressive strength psi 2x4-in. cylinders
		7	14	21	28	
1	High alkali cement 1.00 (1.14 percent as Na_2O) Non-reactive sand 2.25 (Cement content equal to 11 sacks per cu yd)	0.001	0.007	0.012	0.014	6093
2	High alkali cement 0.75 Celite 0.25 Non-reactive sand 2.25 (Cement content equal to 6.1 sacks per cu yd)	−0.014	−0.019	−0.009	−0.007	2790
3	High alkali cement 1.00 Non-reactive sand 90% / Reactive agg. 10% } 2.25 (Cement content equal to 11 sacks per cu yd)	0.036	0.252	0.319	0.384	3767
4	High alkali cement 0.75 Celite 0.25 Non-reactive sand 90% / Reactive agg. 10% } 2.25 (Cement content equal to 6.1 sacks per cu yd)	−0.005	−0.010	−0.003	−0.003	2790
5	High alkali cement 1.00 Non-reactive sand 93.6% / Reactive agg. 6.4% } 4.72 (Cement content equal to 6.1 sacks per cu yd)	0.008	0.010	0.013	0.013	1704

It will be noted from this data that the only reactive combination was Mix No. 3, but that even with the high expansion of this combination, which contained the full amount of cement with no Celite substitution the compressive strength (3767 psi) was higher at 28 days than for the specimens in which Celite had been substituted for the cement (2790 psi) but which did not develop any expansion.

Mix No. 5 consisting of the reduced cement content and no Celite addition tested lower at 28 days (1704 psi) than the corresponding Mix No. 4 with Celite, thereby indicating some beneficial effect of the Celite in a lean mortar mix, as far as strength is concerned.

In conclusion, the primary objective of this contribution is to put additional test data in the published record as a guide to other investi-

gators in a field so replete with apparently contradictory test data and to thereby emphasize the fertility of the field for future study.

At the same time we should not let any apparently contradictory test data related to certain phases obscure the validity of the main thesis, which has withstood attack for ten years at least, namely that one of the most destructive agencies affecting the durability and performance of concrete is some reaction between some mineral or minerals in the aggregate and some constituent of the cement and that this reaction invariably becomes excessively manifest at an early period in the presence of opal or other forms of highly soluble amorphous silica in the aggregate and a high alkali content in the cement.

The fact that ultimate high expansion in the accelerated expansion bar test is frequently observed with relatively low alkali cements, should not be permitted to obscure the fact that California field experience has never disclosed any adverse results when a low alkali cement (less than 0.6 percent) has been used with the same aggregates which reacted adversely with the high alkali (over 0.6 percent) cements.

Furthermore, the early development of excessive expansion in mixtures of reactive aggregates and low alkali cements by the addition of a small percentage of *NaOH* apparently conclusively demonstrates the role of the alkalies in the phenomena.

AUTHORS' CLOSURE

Mr. Mather's discussion emphasizes the very important fact that the reactivity of chalcedony depends not on a single mineral, but upon a mixture of highly siliceous minerals. The anomalies in its behavior as noted by several investigators may, therefore, be the result of its variability in composition. As Mr. Mather observes, much more detailed studies are necessary before the reactivity of a given chalcedony can be quantitatively measured.

Mr. Stanton's contribution giving the details of instances wherein his results differ from those given in this paper is most valuable not only because of his wide experience in this type of investigation, but because it emphasizes the vital necessity of careful control of materials and conditions in experiments on a reaction where so little is known of the basic causes. Even an aggregate consisting of pure opal is variable in water content and in microstructure and, presumably, will vary in its behavior. Mr. Stanton's discussion of the effects of temperature also brings out the fact that different investigators may obtain quite different results because of slight variations in conditions during the test.

The authors did not consider a 25 percent substitution of diatomaceous earth for cements to be practicable. It was simply an additional observation that introduction of pozzolanic materials had a corrective action.

Letters from Readers takes over from the former **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries pertinent to ACI's field of interest . . . A letter without the studied treatment of a major paper, may contain ideas (the most valuable things we have) that get ready access to kindred minds . . . Some of this month's letters are provocative—may bring welcome rejoinders—Editor

Letters from Readers*

BY WAY OF SYNOPSIS

PAUL A. JONES proposes batching bins designed to reduce troubles of contractors, inspectors and control engineers.

HENRY A. WATTS records disappointment in ACI's consideration of "destructive agencies;" how about lactic, fatty and fruit acids, blood, salt and brine, and acid fumes?

EDWARD B. BURWELL, JR. reports andesite aggregate "raising Cain" in Mexico before 1928.

HAROLD A. MUNGER thinks order of combining and mixing time of materials may be important field for further study.

F. O. ANDEREGG wonders how long prestressed concrete will stay that way; what creep is to be expected?

Bins to Reduce "Control" Troubles from Segregation (LR 45-1)

I think a great deal of the trouble in not getting proper results in the field or, let's say, conditions favorable to laboratory findings are due to improper handling of aggregates.

The Bureau of Reclamation has recently taken a step forward in eliminating some of the trouble in requiring that final screening be done at the batching plant. However, if no consideration is given to the design of the batching bins and the contractor is given formal approval of a certain bin, it is rather difficult for an inspector or concrete control engineer to get proper results. It is too easy for material to segregate upon being dumped into bins and come out in very badly graded batches. I am satisfied a remedy for this matter would be to design batching bins

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similar to a honeycomb, made up of tubes between 3 ft and 5 ft in diameter. The number of tubes required would depend upon the capacity desired in the plant. Then if the screening were done directly into these bins, a lot of the inspectors', contractors' and control engineers' difficulties would be eliminated, and about as near possible field conditions meeting laboratory requirements would be accomplished. Segregation would be almost entirely eliminated, and rock ladders could still be used to prevent spalling.

There would no doubt be some objection to this for the reason it would increase the cost of concrete. It is likely that so much better concrete would be produced that some cement could be eliminated and offset the cost. However, it is probable that the increased cost of the additional steel and fabrication would be a very small item compared to the results obtained. A suggestion of this kind might start construction men thinking and eventually get results that have been needed for a long time.

Waldo G. Bowman in his article on 25 years of concrete progress, ("Progress With Concrete, 1923-1948," ACI JOURNAL April 1948, *Proc.* V. 44, p. 963) calls attention to the various outstanding contributions to this progress, including the vibrator, concrete pump, and transit mixer as notable developments by the equipment manufacturer. A much greater contribution could have been made in some kind of equipment to prevent segregation of aggregate before reaching the mixer than anything produced up to date, because the primary requisite is a well-graded aggregate.

PAUL A. JONES

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What to Do about Concrete Deterioration in Food Processing Plants (LR 45-2)

The preliminary announcement of the subjects of the Denver convention stated that one session would be devoted to the resistance of concrete to destructive agencies. This has been one of our major problems, and I had looked forward with much interest to the convention. On the final program, however, I noted that the only destructive agents mentioned were seawater and alkalies, presumably alkalies in groundwater.

There are several other active destructive agencies which constitute a serious problem in industrial building, and I was disappointed to see that the papers to be submitted at the convention cover such a small part of the problem. Following are some of the destructive agencies:

1. Lactic acid—Creameries and dairy plants are faced with a constant repair problem due to the corrosive action of lactic acid from spilled milk or cream.
2. Fatty acids—In meat processing and rendering plants, fats and oils which degenerate into fatty acids cause serious deterioration, particularly of floors. In the rendering departments, fat entrained in the steam from the rendering tanks causes deterioration of the concrete ceilings and columns.
3. Blood—The action of blood on concrete floors is quite similar to that of the fatty acids.
4. Fruit acids—Canneries have encountered rapid deterioration of floors from the spilling of fruit juices.
5. Salt and brine—Rooms used for curing meats, and refrigeration decks above carcass coolers show rapid deterioration from the action of salt on the concrete. We have also seen serious damage from the spilling of brine on floors in freezers. In the case of the refrigeration decks, the deterioration has been accelerated by cycles of freezing and thawing.
6. Acid fumes—In one of our buildings used for the manufacture of commercial fertilizer the concrete roof has deteriorated from the fumes from the acidulating of phosphate rock to such a point that the entire concrete slab must be removed and replaced. Concrete bridges passing over railroad tracks have deteriorated from the action of locomotive gases.

The problem of groundwater alkalies has been practically solved by the development of a new type of cement, and the pitting of concrete pavements from salt has been largely prevented by another new type of cement. Apparently nothing has been accomplished in producing concrete that will resist successfully the destructive agents mentioned above. We have been trying for many years to find a satisfactory wearing surface for concrete floor which would withstand the corrosive action of organic acids when combined with the abrasive action of truck wheels. Thus far we have been unsuccessful.

Practically all the important buildings our company has constructed in the last 35 years have been of reinforced concrete construction, and in that time we have spent millions of dollars in restoration of concrete which has deteriorated due to the action of one or more of the agents listed above. It has been necessary to replace floor surfaces with brick, and the restoration of the structural members has been done mostly by the gunite method.

Will research committees of the American Concrete Institute take note of these problems? We would be glad to hear anything they have to report, as up to now no one has suggested any cure for these troubles.

HENRY C. WATTS

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Andesite Aggregate—Back in 1928 (LR 45-3)

At the suggestion of Byram W. Steele of this office and as a matter of perhaps some historic interest, I quote two paragraphs from a geologic report prepared by me in 1928 on the Mirazalcos Water Power Project in El Salvador, C. A.

> While at Mirazalcos the question of the suitability of crushed andesite as a substitute for sand for gunite and structural concrete was raised, there being no natural sand deposits in the vicinity of the site. I have no first hand information on this, but C. Erb Wuensch, consulting engineer, writes as follows:
>
> "I had some experience with this in Mexico, where it cost us a lot of money learning that andesite has a peculiar habit of weathering with the development of colloidal material which raises Cain with concrete. Another time in Colorado, we made some tests beforehand, and found the same thing to be true, so that I am apprehensive about using anything but a good grade of quartz sand."

This may or may not be the first reference to alkali reactivity. If it is, it was 10 years or more ahead of the times.

EDWARD B. BURWELL, JR.

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Effect of Order of Mixing on Concrete Strength (LR 45-4)

Included herein are data on a series of tests made for the purpose of investigating the influence of order of mixing on the strength of concrete. Previous publications have indicated that this factor may be more important than is generally recognized. (See references at end of letter.)

I do not have enough data, as yet, to justify a formal report on this subject, but believe that it is of such importance that it should be called to the attention of concrete specialists generally in the hope that various investigators may examine its significance under various job conditions.

Four batches of concrete were mixed. Each batch consisted of:

Elgin, Illinois, gravel.....57 lb
Kansas River sand.....56 lb
Type I portland cement.....19 lb
Water.....12 lb

The schedule of mixing was as follows:

A. Cement and water, mix one minute.
Add sand, mix one minute.
Add gravel, mix one minute.

B. Cement and sand, mix one minute.
Add water, mix one minute.
Add gravel, mix one minute.

C. Cement and gravel, mix one minute.
Add water, mix one minute.
Add sand, mix one minute.

D. Cement, sand and gravel, mix two minutes.
Add water, mix one minute.

All aggregate was air dry when placed in the mixer. Slump was 6¼ to 6¾ in. for all batches. Cylinders 6 x 12 in. were made in the usual manner and cured in the moist room for 28 days.

TABLE 1—COMPRESSIVE STRENGTH

Mixing Order	Compressive strength, psi		Ratio, average strength to 16-cylinder average	Rank
	Individual tests	Average		
A	3690, 3160, 2910, 3170	3230	0.97	4
B	3600, 3280, 3030, 3740	3410	1.03	1
C	3520, 3520, 3035, 3195	3320	1.00	2
D	3420, 3260, 2850, 3660	3300	0.99	3

These results (Table 1) are in substantial agreement with data reported by Australian engineers.[2] Obviously, the data are too sketchy to furnish a basis for final conclusions, but they indicate an important field for investigation. It is reasonable to assume that the durability and flexural strength of the concrete may also be affected in similar manner.

As a strictly tentative hypothesis, the writer suggests that in the case of the dry mixed batches the finer particles of cement establish an exceedingly intimate contact with the surface of the aggregate and penetrate into any pores that may exist in the aggregate. When the water is added, it does not entirely displace these cement particles but forms the gel in perfect contact with the aggregate. This results in an

excellent bond. When the cement and water are mixed first there is a greater probability that a water film will form on the surface of the aggregate and prevent the development of a good bond.

Much more work is needed on this problem. It is to be hoped that we will soon have a large collection of data on the numerous questions that immediately suggest themselves.

References:

1. Fitzpatrick, F. L., "Would First Mix Aggregates and Cement Dry," ACI JOURNAL, Feb. 1943, *Proc.* V. 39, p. 312
2. Serkin, W., "Effect of Order of Mixing on Concrete Strength," *Commonwealth Engineer* (Australia), October 1, 1947, reviewed in ACI JOURNAL, May 1948, *Proc.* V. 44, p. 927

HAROLD H. MUNGER

Assistant Professor of Applied Mechanics,
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What will be Prestress Loss Due to Creep? (LR 45-5)

The problem of whether any substantial amount of prestress will remain in a prestressed concrete member where piano wires are prestretched, the concrete poured around them, and then given a certain degree of curing before releasing the wires, is very important.

One of my correspondents, Dr. Ing. Friesecke, in whose former Berlin cement products plant the members for the Berlin Olympic Stadium were fabricated, wrote March 27, 1948:

> "I must also report, that only a completely compacted concrete of the highest quality can supply the grip required for high prestressing. So far this has not been achieved and the experiments of Hoyer have finally failed."

Hoyer, as you may know, wrote a book about 10 years ago on prestressing and is now engaged in publishing another which is expected to "put all other publications on prestressing out of date."

On the other hand, Prof. Otto Graf of the Stuttgart Technische Hochschule in a letter dated May 25, 1948 stated:

> "The reinforcing on the Schäfer slabs creeps very little. This fact has little significance in comparison to the great creepage in pumice concrete (Bimsstein). In the literature sent, you will find that the prestress on the steel has appreciably diminished in the course of time. This decrease is not due to the properties of the steel, but is caused by creep in the concrete."

In manufacturing the Schäfer slabs they use alleyways 100 meters long; pour highest quality concrete from a movable mixer: stretch piano-type wires; pour one slab above another; and after 14 days cut into suitable lengths with a power saw.

Prof. Gustave Magnel's paper on "Creep of Steel and Concrete in Relation to Prestressed Concrete" (ACI JOURNAL Feb. 1948, *Proc.* V. 44, p. 485) rightly emphasizes the importance of the creep properties in steel and concrete, which are far too often disregarded. It is presumed that prestressing at 28 days must have been carried out and against end plates, although the details of the prestressing are not given. It is also to be presumed that the cable was either in a tube or was placed in a slot with later grouting. When rods or wires are initially stretched in the forms and concrete is cast around with them, with release of the wires after a certain degree of hardening, the slip of the steel in the concrete must also be taken into consideration. Since the grip is only a few hundred psi, such slippage may be serious, especially with smooth wires and rods of larger diameter. Proper anchorage of the ends of the reinforcing is extremely important, in addition to securing steel and concrete of very high quality.

F. O. ANDEREGG

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EDITOR'S Note—The following from ACI *News Letter* September 1947, p. 12 may be of interest:

Another outstanding building material found in Germany was the Schäfer slab, a precision-made composite concrete said to be quite different from any material used in the United States or Britain. The slabs were lightweight hollow core pumice concrete covered with a dense concrete coat. Twisted pretensioned wires were embedded in the coating for reinforcement. The slabs were made in 30 and 50 cm widths and in varying thicknesses by an ingenious machine which produced lengths up to 180 m. Pieces of the desired length were sawed from the completed slab. The Schäfer slab was used for floors, roofs and walls. Buildings made from this material with simple mortared joints were found to provide good insulation and resistance to dampness.

Developments in slag cement and light weight concrete manufacture, including the Schäfer slab, are described in *Slag Cements and Lightweight Concretes**.

*Orders for these reports (PB-67458· photostat, $4; microfilm, $2; 59 pages, including photos). should be addressed to the Office of Technical Services, Department of Commerce, Washington 25, D. C., and should be accompanied by check or money order, payable to the Treasurer of the United States.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Examples of the design for prestressed concrete beams

P. W. Abeles, *Concrete and Constructional Engineering*, (London), V. 43, No. 5, May 1948, pp. 149-156 Reviewed by Glenn Murphy

The author presents a number of examples illustrating the theory developed in his article published in *Concrete and Constructional Engineering* for April 1948.

Formulas for the design of prestressed concrete

P. W. Abeles, *Concrete and Constructional Engineering*, V. 43, No. 4, April 1948, pp. 115-122 Reviewed by Glenn Murphy

The author presents a series of simplified formulas developed from a general analysis which he presented in *Concrete and Constructional Engineering* for June, July, and August 1946. The formulas apply to pre-tensioning, post-tensioning, full prestressing and partial prestressing of reinforced concrete beams. They also apply when the members are assemblies of separate blocks. A straight line distribution of stress is assumed throughout.

Formulas are presented for beams containing prestressed reinforcement in one or both faces of the beam.

Examples of the application of the formulas are to be given in later articles.

Design of rigid pavements for heavy wheel loads

Robert R. Philippe, *Civil Engineering*, V. 18, No. 2, February 1948, pp. 32-36 Reviewed by J. R. Shank

This is a discussion of rigid pavements and heavy airplane wheel loads by an engineer closely connected with tests on airfield pavements made during and since the war. The paper is a discussed progress report.

Some uses of prestressed concrete by the Germans during the war

F. Walley, *Concrete and Constructional Engineering*, V. 43, No. 5, May 1948, pp. 137-141 Reviewed by Glenn Murphy

Some of the applications of prestressed concrete in Germany since 1939 are described briefly. Mass production of prestressed reinforced concrete beams permitted the prestressing of several hundred bars at one time. A number of parabolic hog-backed beams designed for loads up to 3000 lb per ft were developed for supporting the concrete roofs of U-boat pens. Bow-string girders nearly 100 ft long are described.

*A part of copyrighted Journal of the American Concrete Institute, V. 20, No. 1, Sept. 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

Methods were developed for the monolithic construction of the sides and bottom of prestressed concrete floats and barges. A number of types of prestressed concrete roof trusses are described. One type with a span of 17 ft weighs only 400 lb.

Particle size of dispersed carbon black affects entrainment of air in concrete

F. J. MARDULIER, *Civil Engineering*, V. 18, No. 3, March 1948, p. 43 Reviewed by J. R. SHANK

Carbon black, often used for coloring concrete has been known to counteract the effect of air-entraining agents. It has been found that this counteraction depends upon the particle size of the carbon black, that above a certain minimum size (not stated) selective adsorption was decreased to the point where its effect was of no consequence. These coarser particle carbon blacks were satisfactory as coloring agents.

Bolt connections for precast reinforced concrete (Bolzenverbindungen fuer Stahlbetonfertigteile)

S. SORETZ, *Oesterreichische Bauzeitschrift*, (Vienna) V. 3, No. 4, April 1948, pp. 57-59
Reviewed by RUDOLPH FISCHL

This article gives the results of tests of steel bolt connections in single and double shear of thin precast reinforced concrete slabs. Suggestions for allowable values for the slabs in bearing and bending are given.

Distribution method of joint rotation (Das Drehwinkelausgleichsverfahren)

O. RAKOSNIK, *Oesterreichische Bauzeitschrift*, V. 3, No. 4, April 1948, pp. 59-61
Reviewed by RUDOLPH FISCHL

This article refers to a distribution method of joint rotations in rigid frames with lateral support. Starting with the relation between joint rotation and stiffness of the adjacent members, this method, similar to Cross's method, distributes and carries over the angular deformations instead of the moments. One example illustrates the method.

Structures for propeller testing beds

Concrete and Constructional Engineering, V. 43, No. 5, May 1948, pp. 133-135
Reviewed by GLENN MURPHY

The propeller testing cells constructed for the DeHavilland Aircraft Co. at Hatfield consist of two hexagonal tunnels each about 200 ft long and 30 ft wide. The sections rest on beams supported by columns resting on foundations, the base of each section being at ground level. The beams and floor slab of the tunnels were cast on a 6-in. sand bed. After the concrete had hardened the sand was removed by water jets so that the structures rested on the columns.

The construction was carried out in two operations, the first involving the casting of the base and two lower sides, the second consisting in casting the upper sides and roof.

Internal dimensions were held to a tolerance of $\frac{1}{8}$ in. Plywood forms framed with steel angles were used.

Several excellent photographs of the construction are shown.

Restoration work prolongs life of concrete structures

A. C. GIESEKE, *Civil Engineering*, V. 17, No. 9, October 1947, pp. 22-26 Reviewed by J. R. SHANK

Seven hydroelectric stations are located at 47 degrees north latitude where most of the concrete surfaces are shaded from the feeble warmth of the winter sun, and mean temperature from November through March is below 32 F with a maximum low temperature of 7.2 F in 76 years.

The cost of restoration was greatest in three of the newest dams and least in the three oldest. The newer structures were built from 1918 to 1925.

The primary faults in the construction of the deteriorating projects are (a) bad aggregate, (b) too much water and segregation and (c) combinations in more moderate degree of both of these faults.

Fully 75 percent of the restoration work was done on the downstream faces of gravity dams. Only ordinary rules for making and placing good concrete where cost of materials is not too important were observed in the restoration work.

Two decades of service prove structural soundness of high-early-strength cement

Civil Engineering, V. 18, No. 4, April 1948, pp. 38-40 Reviewed by J. R. SHANK

Data supplied by the Lone Star Cement Corp. on strength and durability of high-early-strength portland cement concretes indicated that cores cut from a concrete highway at Limedale, Indiana showed 20-year strengths as high as 8200 psi and that later tests indicated continuing increases. Six-bag concretes cured at 50 F showed 4-day strengths of 2400 psi which was 1000 psi stronger than that for ordinary cement concretes.

These data together with observations on the economics of construction brought out the following advantages in the use of high-early portland cement:

1. Savings in form work due to the need for fewer forms, amounting to as much as 60 percent in form material and labor.

2. Benefits to industry and commerce, which include savings of weeks in the completion of factories and plant additions; avoidance of plant tie-ups through rapid rebuilding of floors, driveways, machinery bases, etc.; reduction of slow-order movement during railroad construction and maintenance; and minimum disruption of traffic during street repaving.

3. Time saving for the motoring public due to elimination of much detouring and congestion during highway construction.

4. Better curing resulting in stronger concrete, because the short curing period permits better curing practices under average job conditions.

5. Uninterrupted winter work since with this type of cement, it is only necessary to maintain concrete at 60-70 F for 24 hours, after which period it is serviceably strong and unharmed even by zero weather. Formerly the high cost of prolonged heat protection caused the shutdown of concreting operations in cold weather.

Theory of thin vaults in reinforced concrete

A. L. L. BAKER, *Concrete and Constructional Engineering*, V. 43, No. 4, April 1948, pp. 101-108 Reviewed by GLENN MURPHY

Professor Baker presents an analysis for thin reinforced concrete shells based on the assumption that the member offers negligible resistance to bending and is supported so that the load is distributed by direct stress in the plane of the shell. The analysis is applicable to semi-circular or semi-ellipsoidal domes subjected to distributed loads.

The analysis, based on the membrane theory, leads to a general equation for tangential forces—a general equation containing a function that is dependent upon the shape. The function is evaluated for several special cases.

The author points out the fact that bending stresses may be present under some conditions of loading and support, and that as a result the possibility of buckling should be investigated and stiffening ribs provided if necessary.

The article is accompanied by an editorial containing a discussion of some of the features of this type of construction.

Colne Valley sewerage works

Concrete and Constructional Engineering, V. 43, No. 4, April 1948
pp. 109-114

Reviewed by GLENN MURPHY

Some of the features of the Maple Lodge sewage disposal works near Rickmansworth, Hertfordshire are described.

The project includes approximately 2 miles of tunnels from 6 ft to 6 ft 7¼ in. diameter. They are constructed of precast reinforced concrete segments 2 in. thick and 2 ft long. Three-inch flanges are cast in each segment and are perforated so that the segments may be bolted together. Concrete with a 7-day compressive strength of over 5,000 psi was used and the segments are reinforced with ⅜-in. bars.

After placement the tunnels are lined with concrete followed by two ½-in. coats of gunite. The final coating of gunite includes an acid resistant compound.

A series of sedimentation tanks each 262 ft long 52 ft wide and 12 ft deep are being constructed. The walls and floor are reinforced concrete. The aeration tanks consist of a series of parallel concrete channels 15 ft wide with 15-in. division walls and 6-in. baffle walls. There are 12 settling tanks each 60 ft in diameter with a concrete wall 10 ft high. Sludge digestion tanks and storage tanks about 70 ft in diameter and 28 ft deep are also described. A number of excellent photographs are included indicating various stages of construction.

Photoelasticity, Volume II

MAX M. FROCHT, 505 pp. illustrated, John Wiley & Sons, New York, N. Y., 1948, $10.00

Reviewed by CARL N. ZANGAR

This book is a continuation of the work by Mr. Frocht published in 1941 as *Photoelasticity*, Volume I. Together, the two volumes cover quite completely the published information on two- and three-dimensional photoelastic stress analysis of civil and mechanical engineering structures. Of particular interest are the developments in three-dimensional photoelasticity.

The first two hundred pages of the text are devoted to a comparison of the photoelastic method of experimental stress analysis with certain known analytical solutions. This, perhaps, is somewhat lengthy. However, the solutions, taken from the mathematical theory of elasticity for stress in the semi-infinite plate, stresses in wedges and beams, stresses in disks, and stresses due to a concentrated load in an infinite plate are so well presented that the space allotment seems justified. Throughout the text the author has made his notation and stress convention exceptionally clear, with the assistance of numerous illustrative problems.

The solution giving the stresses in wedges or triangular beams showing the effect of the wedge angle on stresses, and the solution which combines the effect of a concentrated load acting on a beam with the regular bending stresses are clearly and interestingly explained.

Explained in detail are the methods for determining the sum of the principal stresses. These include experimental, graphical, and analytical methods. Experimental methods include the lateral extensometers, interferometers, the membrane analogy, and the method of reflected light for obtaining simultaneously the sum of the principal stresses (isopachics) at all points in the stressed model. The graphical method presented is Neuber's. Chapter 8 contains the analytical methods for determining the sum of the principal stresses. It presents methods for the numerical solution of Laplace's equation. These include algebraic harmonization, the iterative process, the method of block iteration, the method of differences, and the relaxation method. Useful short cuts and examples are given. The procedures given in this chapter have been applied to obtaining the sum of the principal stresses in a two-dimensional body, but some of the procedures

could also be applied to the steady state slow flow of water through granular materials, and to the steady state flow of heat or electricity.

The chapters on three-dimensional photoelasticity will be of most interest to men seeking to develop the photoelastic methods of stress analysis. Much description is given of the freezing method and also the method of scattered light. Fosterite, a new material developed by the Westinghouse Research Laboratories, is also described. This material appears to be well suited to the freezing method of stress analysis, but according to the manufacturer it is not suited to other three- or two-dimensional photoelastic methods of stress analysis. The writer, however, believes that the methods of three-dimensional, photoelastic stress analysis are not as yet developed to a state suitable for complete, general, stress analysis of structures. Urgently needed is an optically sensitive material that could be mixed and cast like gelatin. This would permit the construction of large models containing various shaped openings extending in various directions.

The writer takes one exception with the author, pertaining to the photoelastic interferometer of the Favre type. He states that the instrument is "mainly of scientific and historic significance". This instrument has been made extremely useful in two-dimensional photoelastic analysis. The U. S. Bureau of Reclamation has had such an instrument in operation in its photoelastic laboratory since 1937. It has become the work horse of the photoelastic laboratory, for most problems sent there by the design sections require a complete stress analysis of the structure. This instrument, which analyzes the stresses at one point at a time, gives the magnitude of the separate principal stresses and their directions. An experienced operator can determine the principal stresses and their directions throughout a model in a day or less. In the past bakelite (BT 61-893) models have been used. More recently models have been constructed of Allite, CR-39. This latter material is superior for use in the interferometer due to its flatness and high polish. Models and loading mechanisms for the interferometer can be prepared as rapidly as for the polariscope. Though a minimum amount of time and effort is spent in surveying a model a complete accurate determination of the direction and magnitudes of the principal stresses throughout the structure can be obtained. Recent interferometer analyses have included the determination of stresses in a corbel buttress, in the wall of a pumping plant containing a variety of openings, and around galleries and other openings in dams. Since the introduction of the interferometer, the procedure of obtaining the difference of the principal stress ($(\sigma_1 - \sigma_2)$, the sum of the principal stresses ($\sigma_1 + \sigma_2$), and the direction of the principal stresses in order to obtain a complete stress analysis of a structure has been found unnecessary. For a description of the interferometer the reader is referred to an article by J. H. A. Brahtz and J. E. Soehrens entitled "Direct Optical Measurement of Individual Principal Stresses", *Journal of Applied Physics*, V. 10. 1939,

This book should be of value both to those teaching and to those engaged in the practical use of photoelasticity.

Investigation of formation of cracks in reinforced concrete structures

GEORG WASTLUND AND PER OLOV JONSSON, Swedish Cement and Concrete Research Institute
Proceedings No. 10, 1947 (Stockholm)
AUTHORS' SUMMARY

Formation of cracks in reinforced concrete structures has met with increasing attention since high-grade reinforcement steels with higher allowable stresses came into use. The width of cracks increases with increasing stress in reinforcement, irrespective of the steel grade. Many investigators have grappled with the crack formation problem since the beginning of the thirties, but no satisfactory solution has so far been found.

In the present official Swedish specifications for the design of reinforced concrete structures, the danger of formation of too large cracks has been taken into account. The specifications stipulate that the computed tensile stresses in bending set up in concrete shall not exceed definite allowable values in structures exposed to corrosion. These stresses are to be computed on the assumption that the concrete, in conjunction with the reinforcement ($n = 15$), is able to withstand both tensile and compressive stresses. In this connection, it is to be noted that the above-mentioned allowable tensile stresses are higher than the respective values of the ultimate tensile strength of concrete in bending. It thus follows that the main purpose of the stipulations referred to in the above consists in limiting the extent of cracking, whereas it was not intended that the stipulations should prevent crack formation altogether. From the very beginning it was obvious that the specifications were of preliminary nature and that they would be revised as soon as suitable test results were available.

To contribute to the solution of the crack formation problems an investigation was instituted by the Swedish Cement and Concrete Research Institute in cooperation with the Institution of Structural Engineering and Bridge Building, Royal Institute of Technology, Stockholm, in the summer of 1943.

This investigation was made in order to provide information regarding reinforced concrete bridges and similar structures. The investigation began with tests in the laboratory, followed by records and measurements of cracks in existing bridges. Then a theoretical study of the problem was started in order to deduce formulae for computing the maximum width of cracks. The purpose of the laboratory tests was to study the stresses and deformations due to bond forces between concrete and reinforcement, which are of fundamental importance in this connection, and to examine crack formation in T-beams submitted to positive or negative moments.

The subject of the present paper is confined to empirical data and results concerning crack formation in structural members submitted to positive moments. In the main, this part of the investigation is to be regarded as completed, whereas the study of bond and the examination of crack formation in T-beams subjected to negative moments are still carried on. As a result of the theoretical studies, some formulae, partly empirical, have been deduced for computing the maximum width of cracks in members acted upon by positive moments. Research on these formulae is being continued with a view to providing a more general theoretical basis.

The laboratory tests described in this paper were made in order to examine the effects produced on crack formation in T-beams by the following five factors: (1) diameter of reinforcement bars, (2) ratio of reinforcement, (3) surface properties of reinforcement bars, (4) quality of concrete, and (5) curing conditions.

The total number of the test specimens was 20. The test specimens differed in respect of the factors enumerated in the above. All test specimens had the same theoretical span of 3.0 m. Two equal concentrated loads were applied at ⅓ points of the span. In the first place, crack formation was closely studied during repeated increasing and decreasing loads. The development of cracks was continuously marked and recorded, and the widths of all cracks on both sides of the test beams were measured by means of a microscope.

The results of the laboratory tests are represented in diagrams. Individual results are shown and a comparison of results given. These diagrams seem to indicate that the computed tensile stress in bending set up in concrete is not an appropriate measure of the width of cracks, contrary to what is implied in the Swedish specifications.

Results of the laboratory tests can be summarized as follows:

The width of cracks increases approximately with the diameter of the reinforcement bars.

The width of cracks decreases if the ratio of reinforcement increases. A high ratio of reinforcement in a relatively limited zone close to the beam edge in tension seems to be advantageous.

The use of deformed lug and helical bars, as compared with plain bars, reduces the width of cracks, assuming the diameters of the bars and the stresses set up in the reinforcement to be the same. The width of cracks observed on test beams reinforced with plain bars, 19 mm in diameter, at a stress of 1600 kg per cm^2 in the reinforcement was approximately equal to that measured at a stress of 1900 kg per cm^2 on test beams reinforced with deformed lug bars of the same diameter.

It appears that the quality of concrete does not exert any notable influence on the width of cracks.

Curing in water seems to be slightly more favorable than curing in air, but this tendency is not clearly pronounced.

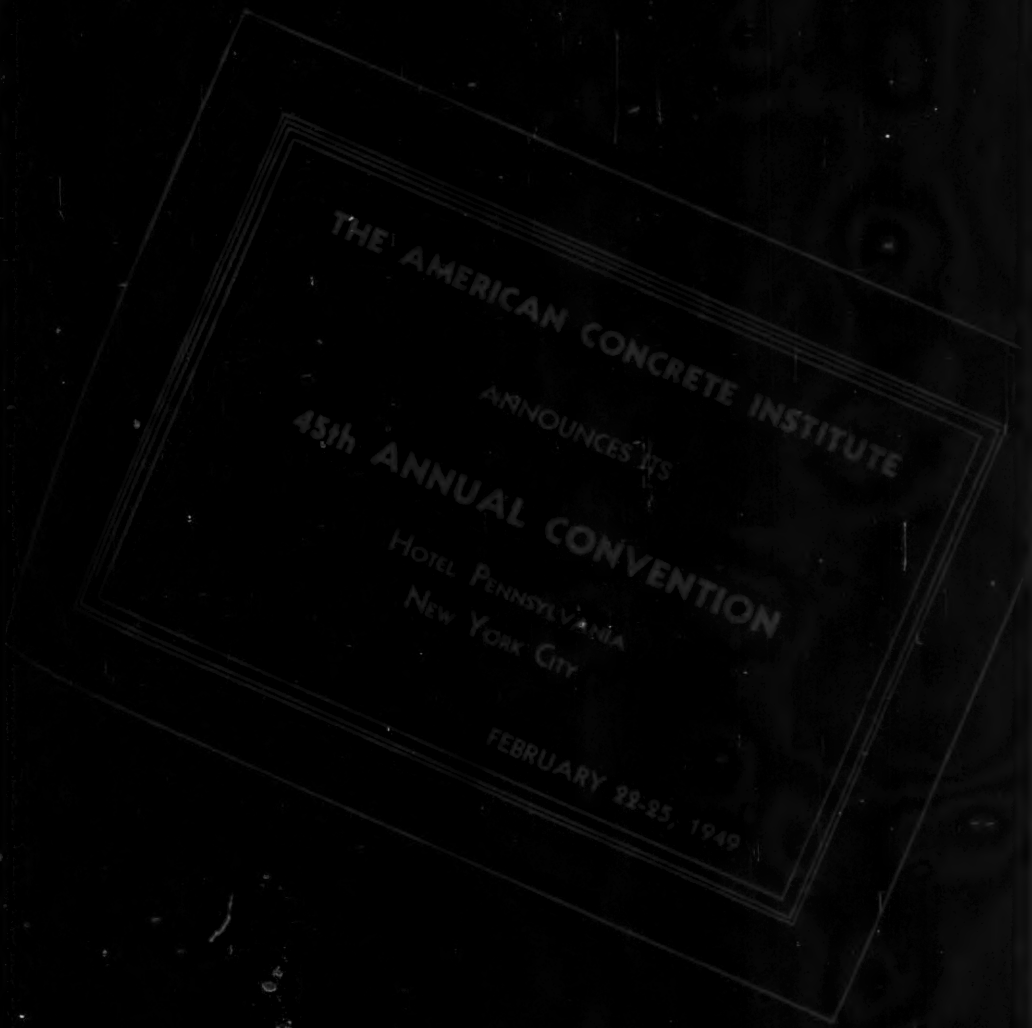
Crack measurements on existing bridges were made on seven Swedish bridges. In general, these measurements covered half the length of each bridge. All concrete surfaces were closely examined, and the cracks were marked, registered and mapped. The width of cracks was measured with an accuracy of about 0.02 mm by means of a microscope. During the measurements of the width of cracks, the bridges were subjected to dead load stresses and possibly also to temperature and shrinkage stresses. Furthermore, the field tests comprised the determination of the temperature and relative humidity of the air and the temperature of concrete on the upper and lower surfaces of the bridge spans.

All bridges showed fairly regular formation of cracks. In the central portions of the spans the cracks were located in the lower parts of the beams. The distance between the cracks was variable. In most cases this distance did not exceed 40 to 50 cm. On continuous bridges no cracks had formed in the vicinity of the zero moment points, but cracks were observed over supports in the upper parts of the beams and in the bridge deck.

The maximum width of tensile cracks observed at the lower edges of the beams was 0.35 mm. It seems, however, that this value has been exceeded at the construction joints where accurate measurements were rendered difficult by the vague contours of the cracks.

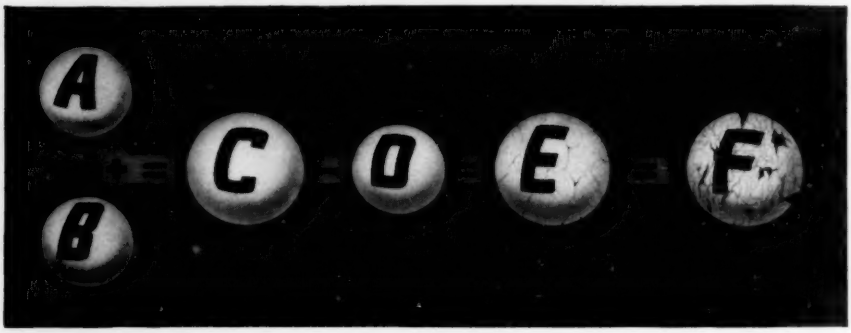
ACI News Letter

Vol. 20 No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1948



PREPAKT REPORTS...

...ON THE HEAT OF HYDRATION IN CONCRETE



TYPICAL CHAIN OF REACTIONS FOUND IN NEW STANDARD CONCRETE MASSES

(A) standard concrete mass *plus* **(B)** heat caused by hydration of materials, *results in* **(C)** expansion of concrete mass as concrete sets. Subtracting **(D)**, shrinkage as heat escapes, *results in* **(E)** cracked concrete caused by restraint, *and yields* **(F)** eventual concrete deterioration.

PREPAKT patented conditioning materials and lower content of hydrating materials operate to minimize the heat produced by hydration. As a result, **PREPAKT** makes possible greater control over heat of hydration, making the concrete mass less subject to the chain of undesirable conditions indicated in the diagram.

Laboratory test data on heat of hydration is shown below:

HEAT OF HYDRATION
(In calories per Gram)

AGE	STANDARD CONCRETE	PREPAKT CONCRETE
1 day	33	17
28 days	84	72

HIGH RESISTANCE TO FREEZING-THAWING • LESS CRACKING
LOWER PERMEABILITY • RESISTANCE TO SULPHATE MOISTURE
HIGH COMPRESSIVE STRENGTH • HIGH BONDING STRENGTH

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ACI

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For the 45th annual ACI Convention - New York February 22-25, 1949

Through meetings of the Technical Activities Committee in April, and again in June, and through the subsequent efforts of individual members of TAC now assigned to develop various subject areas and their program possibilities, the personnel and subject matter of ACI's 45th annual convention—New York City February 22-25, 1949—begin to assume a semblance of outline. Something more definite is expected to emerge by October when TAC and Board of Direction will meet, expecting then to determine the major program features.

Pavement problems

Present indications are that an entire session will be devoted to concrete pavement problems. Due to receive special emphasis among these problems are the effects of variations in materials and construction methods on durability, including the resistance to salt action in winter de-icing, and on the most satisfactory maintenance methods for pavements so affected. This session is to be scheduled in recognition of the serious situation in some parts of the northeast section of this country and elsewhere with respect to durability. Under severe winter conditions some concrete pavements are not giving the service originally anticipated. The increasing use of salt for ice removal and thus far the apparent inability of air-entraining concrete, in some instances, to give full protection against salt scaling, raises the question whether or not air entrainment as now advocated is, after all, the final answer to the problem.

Frank H. Jackson, of the Public Roads Administration, ACI vice president, and a member of TAC, has major responsibility for the pavement session. He is assembling the

papers and discussions through various competent sources at close contact with the problems.

Structural session

Some of the principal elements of a session on structural design are fairly well determined. Nathan M. Newmark, University of Illinois, and Douglas McHenry, Bureau of Reclamation, also members of TAC, share the responsibility in this subject area. Among the prospective contributions is a major one on two-way reinforced concrete slabs. Another covers novel developments in a reinforced concrete framing system in building construction.

Research committee

There will be the usual annual open session of ACI Committee 115, Research—S. J. Chamberlin, chairman, George W. Washa, secretary. This session has been a convention feature of increasing pulling power since its inauguration in 1937. It will consist of a selection of briefly presented material on the techniques of research and preliminary findings of research efforts. These numerous 5- to 15-minute contributions have made interesting sessions in the past; attendance has been large, perhaps because subjects discussed are in the main not yet ready for general and full publicity. Thus attendance at the meeting is almost necessary to get its real value.

General sessions

Two other general sessions are likely to provide opportunity for consideration of several of numerous offerings now "on the fire" or close to the stage of "cooking." These include: lightweight concrete; structural reinforced concrete masonry; masonry unit manufacture; design and execution of architectural concrete; new developments on the subject of bond of steel and concrete; problems of the manufacturers and of the users of ready-mixed concrete; and a simmering down of the voluminous literature of air-entrainment in a brief exposition of the apparent gist of the whole matter.

Aside from one evening for an important dinner session, the New York convention evenings are to be "free" for the individual plans of those attending the series of meetings.

Dates and deadlines

While the convention dates begin with a holiday, February 22, Tuesday, that day will be reserved for administrative meetings of the Board of Direction and of the Technical Activities Committee only.

Technical committees will have their sessions Wednesday morning, February 23 with the first general session in the afternoon of that day—others on Thursday, February 24 and Friday, the 25th, the convention ending in the afternoon of February 25.

Final make-up of the convention program will be strongly influenced this year by what is *available* for TAC advance consideration not later than October—with the object of advance notice to ACI Members and others of what the convention is to include. This too, will encourage the preparation of the liveliest and most informative discussion.

ACI technical committee changes

Important changes in the Institute's technical committee structure will be announced in the October JOURNAL. When the "ACI Directory 1948" went to press as soon after the 1948 convention as possible, there were several items of unfinished business before the Technical Activities Committee—new committee undertakings, reorganizations, new chairmen and other personnel, new assignments or revisions of old.

For the convenience of ACI members the information developed in the approximately half-year interval since closing the current Directory will be brought together in the News Letter pages next month in a form which might be considered as a series of supplements to that part of the Directory, pages 195 to 213.

Who's Who This Month

Committee 604

A new standard "Recommended Practice for Winter Concreting Methods" (ACI 604-48), approved by the Institute's 1948 convention and ratified by ACI members in a letter ballot canvassed June 29, takes first place in this month's JOURNAL, p. 1 of No. 1, V. 20. It represents the work of Committee 604—R. W. Spencer, chairman and his 10 co-workers whose names head up the report in this issue.

Jacob Porter Frankel

Jacob Porter Frankel makes his first JOURNAL contribution with "Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions," p. 21. Mr. Frankel graduated from the University of California, Berkeley, in 1944 with honors in civil engineering, and served 2 years as junior officer on several navy construction regiment projects in the western Pacific.

Following the period of naval duty, he returned to the University of California as research engineer on a project devoted to establishing a scientific basis for the design of artificial limbs. Concurrently, he completed requirements for the M.S. degree in civil engineering, and this paper on mortar strength is abstracted from the thesis for that degree.

Mr. Frankel also taught basic structures and analytical mechanics for the Extension Division of the University of California. Since July, he has been a member of the faculty of the University of California, Los Angeles, where he is doing work toward a doctor's degree. He is a member of Phi Beta Kappa and Sigma Xi.

D. W. Kessler

D. W. Kessler who reports on "Terrazzo as Affected by Cleaning Materials," p. 33, is senior materials engineer and chief of the building stone section, National Bureau of Standards.

Mr. Kessler graduated in civil engineering at the University of Missouri 1908, and received his C.E. degree at George Washington University in 1921. Since 1913 he has worked at the Bureau of Standards in research and testing of various structural materials. Principally interested in durability and methods of durability determination, he has made extensive studies of maintenance problems and has developed many new testing machines and methods of testing. He is the author of some 30 publications on building stone, waterproofing materials and maintenance.

Mr. Kessler is a member of A.S.T.M., A.A.A.S., A.A.E., the Geological Society of Washington, and the Cosmos Club.

R. A. Burmeister

R. A. Burmeister, materials engineer for the City of Milwaukee Testing Laboratory presents evidence in favor of open trucks for hauling ready-mixed concrete. The paper on p. 41, "Transporting Ready-Mixed Concrete in Open Dump Trucks," is his second contribution to ACI literature (see June 1944 JOURNAL which carried "Milwaukee's Experiences with Air-Entraining Cements").

Born in Milwaukee in 1906, Mr. Burmeister graduated from the University of Wisconsin in 1928 and entered the employ of the City of Milwaukee Testing Laboratory. In 1930 he became assistant in charge of the materials testing laboratory of the Wisconsin State Highway Commission. In this capacity, he participated in numerous tests on concrete and mix design covering several hundred Wisconsin aggregate sources. From these and later tests he says he formed a "dim view" of the practical value of the water-cement ratio law. It was Mr. Burmeister who

was quoted in the April News Letter, as saying that the water-cement ratio law is "a cloud through which we are trying to peer at the true qualities of concrete."

In 1935 he transferred to the Milwaukee division of the highway commission, where he did field work on concrete and soils. He returned to the Milwaukee testing laboratory in 1942 to assume his present position. He is also consultant to the Milwaukee Central Board of Purchases on specifications, research and tests on a wide variety of materials. An ACI member since 1929, Mr. Burmeister is also a member of A.S.T.M., A.S.C.E. and A.S.M.

T. M. Kelly, Louis Schuman and F. B. Hornibrook

Three authors report on work performed while they were all employed by the National Bureau of Standards; the paper is "A Study of Alkali-Aggregate Reactivity by means of Mortar Bar Expansions," appearing on p. 57 this month. The extensive study indicates that diatomaceous earth added as a cement replacement halts much dangerous expansion.

T. M. Kelly, now a concrete technician for the operations division, Corps of Engineers, Portland, Oregon, District, was born in upstate New York, and attended Union College at Schenectady. He graduated in civil engineering in 1933 and for 3 years worked for various federal and municipal agencies on field surveying, street and sewer projects. During 1936 and 1937 he was a calculating engineer with the American Locomotive Co., Schenectady. In 1937 he entered the employ of the National Bureau of Standards as materials engineer in the masonry construction section. He served in a similar capacity successively in the cement reference laboratory and the concreting materials section until April 1948 when he assumed his present duties.

During the period of employment at the National Bureau of Standards, Mr. Kelly spent a 3-year military leave on active duty with the naval reserve as an aircraft overhaul and repair officer. This activity included a concentrated course in aeronautical engineering at California Institute of Technology and overseas duty on Guadalcanal and Green Island.

Louis Schuman was born in Manchester, England, came to the United States in 1910 and attended school in Philadelphia. He received a B.A. degree from Temple University in 1925 and did graduate work in physical chemistry at Massachusetts Institute of Technology. Author of several research papers on properties of cement and concretes, including tensile and wear testing, he has been with the National Bureau of Standards since 1926. During World War II he was in the ordnance development division and assisted in development of the magnetic fluid clutch. Mr. Schuman is now employed as engineer, test section, organic and fibrous materials division of the bureau.

He is a member of the American Jewish Congress and a former member of ACI and the American Chemical Society.

F. B. Hornibrook, native of Washington, attended Willamette University and the University of Oregon, graduated from the State College of Washington in 1930 and went at once to the National Bureau of Standards where he served for 16 years. He received his master's degree in physical chemistry, University of Maryland 1934.

While at the Bureau of Standards, Mr. Hornibrook worked on calorimetry of cement, development of an apparatus for measuring the vibration in concrete during compaction, and the development of a sonic apparatus for measuring the dynamic modulus of elasticity of concrete. He was author or co-author of many technical reports on these developments, has been active in technical committee work of ACI and A.S.T.M. and has contributed to ACI *Proceedings* and discussions. In 1946 he joined the Master Builders Co., Cleveland, Ohio, becoming director of research, a position which he left to join the Best Fertilizer Co., Oakland, Calif., in 1947.

People, Projects and Plaudits

Kavanagh advances at N.Y.U.

Dr. Thomas C. Kavanagh, ACI member on the staff of New York University, has been promoted to the rank of assistant professor in the aeronautical engineering department.

Dutch interested in oil wells

The May 1948 issue of *Polytechnisch Tijdschrift*, from the Netherlands, contains an almost complete translation of a paper entitled "Oil Well Cementing Practice" by R. E. Moeller and Hayden Roberts, which was presented at the ACI convention Cincinnati, February 24, 1947.

From the Netherlands

Among ACI's June visitors from abroad were J. A. F. C. Marckman and J. DeVries representing Bredero's Bouwbedrijf N. V. of Utrecht, Holland. Messrs. Marckman and DeVries were making an extensive tour of the United States, with a special interest in securing information on lightweight concrete. Mr. DeVries is a new member of the Institute.

Hanna wins Thompson award

At the A.S.T.M. meeting in Detroit, June 21-25, W. C. Hanna, chief chemist for the California Portland Cement Co. received the Sanford E. Thompson award to the author of a paper of outstanding merit in the field of concrete and concrete aggregates. His 1947 paper was "Unfavorable Chemical Reactions of Aggregates in Concrete and a Suggested Corrective." Mr. Hanna is a member of the Institute as a representative for his corporation.

Maguire gets science doctorate

The College of the Holy Cross conferred the honorary doctor of science degree on Charles A. Maguire, Providence, R. I., at its 102nd commencement June 9, 1948. Mr. Maguire, who graduated from Holy Cross in 1909, is a consulting engineer and head of the engineering group that recently compiled the traffic report of metropolitan Boston and the expressway system report for metropolitan Providence He became a member of the Institute in 1944.

A. T. Rogers to Lehigh Materials Co.

ACI member A. T. Rogers has joined the Lehigh Materials Co., Philadelphia, (until recently The Lehigh Navigation Coal Co.) as manager of sales engineering. He was previously manager of the technical service department of the North American Cement Corp., New York, N. Y. The Lehigh company recently began production of a lightweight aggregate made from carbonaceous metamorphic shale under the trade name *Lelite*.

Straub heads hydraulics research group

Lorenz G. Straub, director of the St. Anthony Falls (Minnesota) Hydraulic Laboratory and head of civil engineering at the University of Minnesota, was elected president of the International Association for Hydraulic Structures Research at the association's meeting in Stockholm, Sweden in June.

Dr. Straub spent several months in Europe lecturing and conferring with engineers in France, Germany, Austria, Italy, Switzerland and the United Kingdom. In Paris he spoke at the centennial meeting of the French Society of Civil Engineers and addressed the Association des Ingenieurs-Docteurs de France at the Sorbonne. Dr. Straub has been an ACI member since 1926.

Cement industry award

Five of the six cement companies qualifying for "highest merit awards" citations in an annual report survey conducted by *Financial World* are members of the American Concrete Institute. Stockholder reports of these companies will be studied as a basis of choosing one of these the "best of the Industry" to be presented at the *Financial World* Annual Report Awards Banquet October 21 in New York. The six companies cited are: Bessemer Limestone and Cement Co., Lone Star Cement Co., General Portland Cement Co. Lehigh Portland Cement Co., Marquette Cement Manufacturing Co., and Missouri Portland Cement Co.

Prestressing

A. E. Cummings, director of research, Raymond Concrete Pile Co., New York, N. Y., has taken over the chairmanship of ACI Committee 323, Prestressed Reinforced Concrete following resignation of A. J. Boase. Mr. Cummings, a University of Wisconsin graduate, and ACI member since 1924 has written numerous technical papers on soil mechanics and foundation engineering. In addition to A.S.C.E. and A.S.T.M. affiliations, he is a member of the Boston Society of Civil Engineers, the Society for Experimental Stress Analysis and Sigma Xi.

Other members of Committee 323 are D. E. Parsons, Washington, D. C., Ivan Rosov, New York, N. Y., Herman Schorer, Valhalla, N. Y. and H. B. Zackrison, Washington, D. C.

Frank H. Jackson honored by A.S.T.M.

Frank H. Jackson, principal engineer of tests, Public Roads Administration, was awarded honorary membership in the American Society for Testing Materials at its annual meeting in Detroit, June 21-25, 1948. Mr. Jackson, who is now vice president of the American Concrete Institute and who has been for some time very active in Institute work, is likewise a long-time, active member of A.S.T.M. He studied at Ohio State University before accepting a position with the Bureau of Public Roads (now P.R.A.) in 1905, where he has served continuously ever since. His primary interest has been testing, and has included both research and standardization in this field. Since 1915, he has been closely associated with investigations of non-bituminous road materials.

Mr. Jackson has been author or coauthor of seven Institute papers, has served on advisory, publications and technical activities committees. He had served five years on the board of direction before his recent election as vice president. His most recent contribution to ACI literature (with W. H. Allen) was "Concrete Pavements on the German Autobahnen," June JOURNAL, p. 933.

First concrete for P.C.A. laboratory

First concrete for the Portland Cement Association's new two million dollar research and development laboratory was placed June 30, on the site where construction is beginning in the village of Skokie, 15 miles northwest of Chicago.

The lever which started the mechanism to allow the first cubic yard of concrete for the new structure to fall into place was released by Charles E. Aspdin of Hasbrouck Heights, N. J., the great-great-grandson of Joseph Aspdin who invented portland cement in Leeds, England, in 1824.

Others who participated in the ceremony were Frank T. Sheets, P.C.A. president; W. A. Wecker, president of Marquette Cement Manufacturing Co., S. W. Storey, president of the General Portland Cement Co. and the Consolidated Cement Corp.; and W. C. Russell, president of Peerless Cement Corp.; all directors of the Portland Cement Association.

Completion of the buildings, which are being erected by the Turner Construction

Co., with Carr & Wright as architects and engineers, is expected within a year. They will contain more than 25 specialized laboratories, including rooms capable of maintaining temperatures ranging from 20 below zero to 130 F.

Savage advises India on a bigger Hoover Dam

The new national government of India is initiating work on a dozen or more hydroelectric schemes which will provide approximately four million kilowatts of electric energy to numerous industrial plants in India which are being set up or will be set up in future, to operate pumping machinery for irrigation and drainage, and for domestic use. In addition to power generation, high-level irrigation and navigation are other objectives.

The most spectacular project is the dam across the Kosi river near Barahkshetra in Nepal territory. Dr. J. L. Savage, ACI member since 1929 and Turner medal winner in 1946, visited the site recently to advise Indian engineers. The dam will rise 750 to 850 ft above bedrock to be the highest dam in the world (Hoover Dam is 730 feet). The power station at the dam, with a 1.8 million kw capacity, will extend electricity to every village and home in most parts of Bihar and Nepal, and leave ample reserves for large-scale industrial development.

In addition, it will make possible irrigation of one million acres of land in Nepal and two million acres in Bihar, thus providing an annual addition of one million tons to the food production of the area. Added advantage is that the river channel, which has changed its course nine times and has devastated approximately 4,000 sq mi of land in Nepal and Bihar, will be stabilized and made fit for navigation throughout the year. The construction of the dam is expected to take 10 years and cost about $300,000,000.

Wessman new dean at University of Washington

Harold E. Wessman, prominent civil engineering educator and Institute member since 1936, has accepted appointment as dean of the College of Engineering of the University of Washington, Seattle. Dr. Wessman leaves his position as chairman of the Civil Engineering Department at New York University, where he has been for the past 11 years.

His engineering career began at the University of Illinois where he received bachelor's and master's degrees and a professional degree in civil engineering. Following several years of structural practice and some teaching at the University of Illinois, in 1929 he was appointed associate professor of bridge engineering by the Chinese Ministry of Railways at Chiao-Tung University, Shanghai. He returned from the Orient in 1930 to serve first as a structural engineer for a New York consulting firm, then as water commissioner and rate consultant in Rockford, Ill. In 1933 he was named associate professor of civil engineering at the University of Iowa, and in 1936 he received his doctorate at the University of Illinois.

New York University selected Dr. Wessman to head its Civil Engineering Department in 1937 and while directing that program he served as special consultant to many engineering concerns. He developed an evening graduate program in structural engineering at New York University which has given advanced training to many engineers in the metropolitan area. He is a past president of the American Institute of Consulting Engineers.

Teller heads strong ACI committee on pavements

Formation of a new technical group, Committee 325—Structural Design of Concrete Pavements for Highways and Airports, has just been announced by the executive committee of the ACI board of direction. L. W. Teller of the Public Roads Administration, Washington, D.C. will head the group assigned to the development of a recommended practice for the design of structural details for concrete highway and airport surfaces. Such

factors as slab dimensions, cross-section, joints and reinforcement will be considered on the basis of the best available information, and new developments will be constantly considered in order to keep recommended practices up to date.

Men who have accepted appointment to the new committee include: Henry Aaron, Civil Aeronautics Administration, Washington, D. C.; E. A. Finney, Michigan State Highway Department, Lansing; Bengt Friberg, Granite City Steel Company, Granite City, Ill.; and A. T. Goldbeck, National Crushed Stone Association, Washington, D. C. Other new members accepting appointment are: T. J. Kauer, Wire Reinforcement Institute, Washington, D. C.; H. L. Lehmann, Louisiana Department of Highways, Baton Rouge; P. E. Masheter, Ohio State Department of Highways, Columbus; N. M. Newmark, University of Illinois, Urbana; and G. S. Paxson, Oregon State Highway Commission, Salem.

William Van Breemen, N. J. State Highway Department, Trenton; C. A. Willson, American Iron and Steel Institute, New York, N. Y.; K. B. Woods, Purdue University, West Lafayette, Ind.; and Frank N. Wray, Highway Research Board, Washington, D. C.

Crum wins Marston medal

Roy W. Crum, director of the Highway Research Board and past president of the American Concrete Institute, received the Marston Medal at June 1948 commencement exercises at Iowa State College. This award, established through the inspiration and generosity of Anson Marston, dean emeritus of engineering at Iowa State, is presented to an engineering alumnus of the college, in recognition of outstanding achievement in engineering. Not only technical design and construction are considered, but accomplishments in planning, development, and management of manufacturing, public utilities and public works enterprises are recognized. Achievements in research and education as well as activity in the national professional engineering societies are considered in selecting the awardee.

Roy Crum graduated in civil engineering at Iowa State in 1907, and received his professional degree there in 1914. He first worked for the Pennsylvania Railroad, and then joined the faculty of Iowa State College where he was an instructor and later associate professor and structural engineer for the Engineering Experiment Station. It was during this period that he conducted research on materials for the "new" concrete roads which were then just coming into use. His mix designs using the screened and unscreened gravel of Iowa deposits were a major contribution to the economic feasibility of improving the highways of that region. He also was associated with Anson Marston in the investigation of earth pressures on underground conduits.

In 1919 he became engineer of materials and tests for the Iowa State Highway Commission, and his organization efforts made this department one of the outstanding of its kind in the country. Cooperating with fellow officials in other states, he led in the development of standard methods for tests of highway materials, and in directing research involving all types of highway materials. In 1928 he was appointed director of the Highway Research Board, where for the past twenty years he has been responsible for the administration of technical and publications activities of that organization.

Mr. Crum's active participation in technical society affairs has been widely diversified. An ACI member since 1924, he has served on advisory, publications, program and standards committees; as director, vice president, president and past president, he served 9 years continuously on the ACI board of direction. He is now a member of the A.S.C.E. board of direction. He has been active in the American Society for Testing Materials and the Iowa Engineering Society, and is a member of the American Association for the Advancement of Science, Sigma Xi, Tau Beta Pi and Phi Kappa Phi.

New Members

In view of the Journal's summer hiatus—no issues in July and August—the list of new members which follows (applications received in May, June and July) contains 246 names, listed alphabetically, by states and countries.

You can see readily how your own home state or country is doing in its contribution to ACI growth.

It is interesting that 23 percent of the 246 new members are from outside U.S.A.; also that 16 percent of the U.S. total are from California (see Honor Roll, p. 17).

Former listings have been by membership classes—Contributing, Corporation, Individual, Junior, Student. These classifications are now shown in parentheses following the name of each new member: (Cont.), (Corp.), (Indiv.), (Jr.), (St.).

Net increase of membership from July 31, 1947 to July 31, 1948 is 729 for a total of 4243.

What part might you play in ACI's continued growth!

Alabama

Steneck, Eugene (Indiv.) 5 Norman Drive, Mountain Brook, Birmingham 9, Ala.

Arizona

McLeod, Neil R. (Indiv.) 3010 N. 11th Ave., Phoenix, Ariz.

Arkansas

Bryant, Eugene (Jr.) Corps of Engineers, Little Rock District, 300 Broadway St., Little Rock, Ark.

Skinnarland, Einar (Indiv.) c/o Ozark Dam Constructors, Mountain Home, Ark.

California

Barhite, Richard George (St.) 750 Pine St., Apt. 46, San Francisco, Calif.

Barieaù, Jerome Eric (Indiv.) 185 Orange St., Auburn, Calif.

Beatty, Lynn (Indiv.) Western Gravel Co., P. O. Box 1026, Campbell, Calif.

Blake, Robert C. (St.) 1669 Oxford St., Apt. 4, Berkeley 9, Calif.

Chace, Clyde B. (Indiv.) 31 Sutter St., San Jose 10, Calif.

Collins, Robert Lewis (Jr.) 307 N. Almansor St., Alhambra, Calif.

Coolidge, James Douglas (St.) 737 E. 23rd St., Oakland 6, Calif.

Desert Materials Corp. (Corp.) 6371 Wilshire Blvd., Los Angeles 36, Calif. (L. J. Finch)

Dubbs, C. C. (Indiv) 1314 E. Chapman, Orange, Calif.

Finlayson, Donald J. (St.) 4516 Coldwater Canyon, North Hollywood, Calif.

Flores, Ambrosio R. (Indiv.) Manila Dist., Manila Sub-Office, Corps of Engrs., Dept. of Army, APO 900, c/o Postmaster, San Francisco, Calif.

Gabelman, William E. (Indiv.), c/o Dept. of Water & Power, City of Los Angeles, 405 S. Hill St., Los Angeles, Calif.

Gould, John J. (Indiv.) Financial Center Bldg., San Francisco, Calif.

Helsley, J. T. (Indiv.) 4215 Bandini Blvd., Los Angeles 23, Calif.

Helstrom, Robert (Indiv.) 945 Lenzen Ave., San Jose, Calif.

Keith, A. C. (Indiv.) Court House, Riverside, Calif.

Kummert, Walter H. (Indiv.) 3424 Cecil St., San Jose, Calif.

Leininger, Carl J. (Indiv.) City Hall—Room 14, San Jose, Calif.

Levy, Edwin F. (Indiv.) Div. of of San Francisco Bay Toll Crossings, 74 New Montgomery St., San Francisco, Calif.

Lewis, Morgan M. (Indiv.) c/o California Polytechnic College, San Luis Obispo, Calif.

Lewis, Richard E. (Jr.) 1080 Elm Ave., Long Beach, Calif.

Lewis, Ronald E. (Indiv.) Transit Mixed Concrete Co., 3464 E. Foothill Blvd., Pasadena 8, Calif.

Lewon, R. (Indiv.) Paraffin Companies, Inc., Research Dept., 1550 Powell St., Emeryville, Calif.

Lyons, S. P. (Indiv.) 1805 Courtney Ave., Los Angeles 46, Calif.

McDowell, James Martin (St.) 1799 Euclid Ave., Apt. 7, Berkeley 9, Calif.
McKittrick, E. S. (Indiv.) 2672 Flower St., Huntington Park, Calif.
Peterson, Jerome L. (Indiv.) 2922 S. Sunol Dr., Los Angeles 23, Calif.
Riggs, Louis William, Jr. (St.) 2636 Etna St., Berkeley 4, Calif.
Steinbrugge, John Max (Jr.) 575 Cherry Ave., Long Beach 12, Calif.
Townsend, Thomas Harold (Indiv.) 991 Second St., Napa, Calif.
Will, Jake (Indiv.) 824 E. Cook St., Santa Maria, Calif.

Colorado

Bastedo, Walter A. (Indiv.) 935 Locust, Denver 7, Colo.
Fattor, Arthur P. (Indiv.) 4358 Steele St., Denver 16, Colo.
Hester, Pasqual Edwin (St.) P. O. 627, Louisville, Colo.
Montagne, Paul A. (Indiv.) 847 Pearl St., Denver 3, Colo.
Satterthwait, Don R. (Jr.) 2229 9th Ave., Greeley, Colo.
Youngclaus, George Dunton (St.) 2204 Hudson St., Denver, Colo.

Connecticut

Burdick, John J. (St.) 550 Woodstock Ave., Stratford, Conn.

Delaware

Tyndall, Joseph Thomas (St.) R. D. No. 3, Georgetown, Del.

District of Columbia

Foster, Bruce E. (Indiv.) National Bureau of Standards, Washington 25, D. C.
Locraft, Bernard F. (Indiv.) 1179 New Hampshire Ave., N. W., Washington 7, D. C.
Teller, L. W. (Indiv.) Public Roads Administration, Washington 25, D. C.
Thompkins, Francis M. (Indiv.) Chas. H. Tompkins Co., 907 16th St., N. W., Washington, D. C.
Umar, B. S. (Jr.) c/o Educational Liaison Officer, 2111 Massachusetts Ave., N. W. Washington 8, D. C.
Widmayer, John J. (Indiv.) Denrike Bldg., 1010 Vermont Ave., N. W., Washington, D. C.

Florida

Allender, Howard F. (St.) 659 40th Ave. S., St. Petersburg, Fla.
Ross, Thomas C. (Indiv.) Florida Crushed Stone Co., Box 608, Ocala, Fla.
Spangler, Byron Dement (Indiv.) Apt. 56-P, Flavet No. 1, Gainesville, Fla.

Georgia

King, Lavelle R. (Indiv.) 320 Boatner Ave., Cartersville, Ga.
Shiffler, Charles W. (Indiv.) 310 Boatner, Cartersville, Ga.

Illinois

Coleman, George (Jr.) 5309 Woodlawn, Chicago, Ill.
Corrigan, Edward (St.) 1822 S. Gunderson Ave., Berwyn, Ill.
Du Boff, Dean M. (Jr.) 515 E. Virginia Ave., Peoria, Ill.
Goldberg, Bertrand (Indiv.) 721 N. Michigan Ave., Chicago 11, Ill.
Jacabsen, J. B. (Indiv.) 5202 Cornell Ave., Chicago 15, Ill.
Katz, Hyman H. (Indiv.) c/o Allied School of Mechanical Trades, 1340 S. Michigan Ave., Chicago 5, Ill.
Koehler, Alexander C. (St.) 3125½ S. Harlem Ave., Berwyn, Ill.
Kuston, LeRoy Alfred, (St.) 2037 N. La-Crosse Ave., Chicago 39, Ill.
Mass, Marvin L. (Indiv.) 462 Washington Blvd., Oak Park, Ill.
Risberg, Robert J. (St.) Depue, Ill.
Wu, Tien-Hsing (St.) 311 W. Illinois St., Urbana, Ill.

Indiana

Hagerman, Theodore F. (Indiv.) 402 E. Superior St., Fort Wayne 2, Ind.
Nelson Engineering Sales, Inc. (Corp.) 226 J. M. S. Bldg., South Bend 5, Ind. (Carl A. C. Nelson)

Iowa

Huang, Jung-Han (St.) C-16, Quadrangle, University of Iowa, Iowa City, Ia.

Kansas

Green, Mont J., Jr. (Indiv.) Ulrich Bldg., Manhattan, Kans.
Jarrett, Robert Patterson (St.) 415 S. Pennsylvania, Independence, Kans.

Louisiana

Grobe, William Charles, Jr. (St.) Box 91, Louisiana Tech Station, Rushton, La.
Jeffrey, Neill P., Jr. (St.) 120 Audubon Blvd., New Orleans, La.

Maine

Smith, Sheldon Donald (St.) 93 Mt. Pleasant Ave., Skowhegan, Maine

Maryland

Colony, David C., Jr. (St.) 4312 Kaywood Dr., Apt. 2, Mt. Ranier, Md.
Scowe, N. D. (Indiv.) Muirkirk, Md.
Thompson, Joseph T. (Indiv.) 3802 Juniper Rd., Baltimore 18, Md.

Massachusetts

Bailey, Wayland S. (Indiv.) Box 37, Norwell, Mass.
Cavanaugh, David J. (Indiv.) 5 Wolcott St., Holyoke, Mass.
Olaechea, Jaime (Jr.) 325 Commonwealth Ave., Boston, Mass.

Michigan

Abdulahad, Albert Abdullah (Jr.) 1227 S. University Ave., Ann Arbor, Mich.
Anderson, William F. (St.) 184 Washington St., Manistee, Mich.
Arocha, Alberta Aranda (St.) 543 S. Division, Ann Arbor, Mich.
Atherton, Neil Pierce (St.) 2915 Hogarth, Apt. 4, Detroit 6, Mich.
Fletcher, Douglas Wellington (Indiv.) 601 Woodland, Detroit 2, Mich.
Nieh, Tseng Lu (St.) 1109 Willard St., Ann Arbor, Mich.
Qalla, Mohmoud (St.) c/o International Center, University of Michigan, Ann Arbor, Mich.
Schaaf, Cecil F. (Indiv.) 2521 Stevenson, Flint, Mich.
Vranich, Emil F. (St.) 535 S. Division St., Ann Arbor, Mich.

Minnesota

G. T. Tennant Co. (Corp.) 2530 N. Second St., Minneapolis 11, Minn. (Lloyd Hale)

Mississippi

Oden Construction Co. (Corp.) 602 S. Gallatin St., Jackson, Miss. (C. G. Oden)

Missouri

Jantzen, Louis P. (St.) 4256 Shenandoah Ave., St. Louis 10, Mo.
King, Thomas Arthur (St.) 306 Hitt St., Columbia, Mo.
Lockett, Harold B. (Indiv.) c/o Horner & Shifrin, Consulting Engrs., 803 Shell Bldg., St. Louis 3, Mo.
Smith, Alfred T. (Indiv.) 5740 Lansdowne Ave., St. Louis 9, Mo.

Montana

Newcomer, A. W. (Indiv.) U. S. Bureau of Reclamation, P. O. Box 517, Helena, Mont.

Nebraska

Theisen, Casper J. (St.) Osmond, Pierce County, Nebr.
Vancura, O. J. (Indiv.) 1141 N. 11th St., Omaha, Nebr.

Nevada

Woodgate, Alfred Melvin (St.) 510 S. Minnesota St., Carson City, Nev.

New Jersey

Anderegg, Frederick O. (Indiv.) John B. Pierce Foundation, Raritan, N. J.
Camlet, J. Thomas (Indiv.) Hughes Bldg., 667 Main Ave., Passaic, N. J.
Karl, William Wainright (Jr.) 227 Grove St., Montclair, N. J.
Larned, Arthur Thomas (Indiv.) 94 Home Ave., Rutherford, N. J.

New Mexico

Clough, Richard H. (Jr.) 606 N. Girard Ave., Albuquerque, N. M.

New York

Allison, W. H. (Indiv.) Clarkson College of Technology, Potsdam, N. Y.
Andrews, Donald C. (Indiv.) 15 Downer Ave., Scarsdale, N. Y.
Baxter, J. F. (Indiv.) 73 Longview Rd., Port Washington, N. Y.
Campbell, M. Douglas (St.) Apt. 14—Vets Village, Canton, N. Y.
Creidenberg, Carl (St.) 138-30 77th Ave., Flushing, L. I., N. Y.
Eipel, William (Indiv.) 101 Park Ave., New York, N. Y.
Fabian, Joseph L. (St.) 190 Bay St., Staten Island 1, N. Y.
Hogan, Frank (Indiv.) Design Service Co., Engineering Div., 50 Broad St., New York, N. Y.
Hogan, John J. (Indiv.) 430 E. 148 St., New York 55, N. Y.
Jonap, Robert D. (Indiv.) Technical Service Dept., North American Cement Corp., 41 E. 42nd St., New York 17, N. Y.
Kao, Wayne K. (Jr.) 601 Cathedral Parkway, New York 25, N. Y.
Lund, B. D., Jr. (Indiv.) 215 W. 23rd St., New York 11, N. Y.
Lyons, Stanley B. (Indiv.) P. O. Box 304, Peekskill, N. Y.
Pare, Robert Lee (Jr.), 21 Cottage Park, Weedsport, N. Y.
Preload Enterprises, Inc. (Corp.) 420 Lexington Ave., New York 17, N. Y. (Curzon Dobell)
Reetz, John Henry (Indiv.) 147 W. Lena Ave., Freeport, N. Y.
Stolle, Frank (Indiv.) 7226 72nd Court, Brooklyn 9, N. Y.
Winarski, L. (Indiv.) 530 E. 6th St., New York 9, N. Y.
Zollman, Charles C. (Indiv.) 77-54 Austin St., Forest Hills, L. I., N. Y.

North Carolina

Gurley, Henry T. (Indiv.) Box 1233, Burlington, N. C.
Scott, C. B. (Indiv.) Box 694, Hickory, N. C.

Ohio

Carroll, Robert James (Indiv.) 145 Blenheim Rd., Columbus 2, Ohio
Detzel, George E. (Indiv.) 2303 Gilbert Ave., Cincinnati 6, Ohio
Killin, Robert L. (Indiv.) 504 Irving Ave., Dayton 9, Ohio
Kuo, Shan Sun (St.) 1122 River Road Dormitories, Ohio State University, Columbus 10, Ohio
Schmidt, Leo W. (Indiv.) 10000 Granger Rd., Cleveland 5, Ohio
Ullberg, Erik (Indiv.) Ohio River Div. Laboratories, Corps of Engrs., U. S. Army, Mariemont, Cincinnati 27, Ohio
Wagner, N. O. (Indiv.) Portland Cement Association, 50 W. Broad St., Columbus Ohio

Oregon

Anderson, Joe Meeks (St.) 1486 State St., Salem, Ore.
Edgerton, Roy Clarence (Indiv.) 1067 Edgewater St., Salem, Ore.
Schwarz, Walter A. (Indiv.) Dorena, Ore.

Pennsylvania

Artuso, Joseph F. (St.) 964 Thompson Ave., Donora, Pa.
Central Builders Supply Co. (Corp.) Island Park, Sunbury, Pa. (A. B. Markunas)
Davis, Paul Marsh, Jr. (St.) 403 E. Cranberry Ave., Hazelton, Pa.
Gokyigit, Ali Nihat (St.) 108 Boas St., Harrisurg, Pa.
Shope, Henry Andrew, Jr. (St.) 148 W. Marietta St., Munhall, Pa.

South Dakota

Cornelius, Henry Allen, Jr. (St.) Canton, S. D.
Rogness, Aaron Nicholas (St.) Astoria, S. D.
Vaudrey, Calvin (St.) 716 E. Clark St., Vermillion, S. D.

Tennessee

Bryan, Albert Henry, Jr. (St.) 321 Woodland St., Watertown, Tenn.
McDougle, Edwin Andrew (Indiv.) 910 Walker St., Elizabethton, Tenn.
McVean, Charles A. (Indiv.) Corps of Engineers, Memphis Dist., Box 97, Memphis, Tenn.
Quickel, William A. (Jr.) 2012 Saxton St., Knoxville, Tenn.

Texas

Brooks, John Albert (St.) 4211 Gilbert Ave., Dallas 4, Texas
Campbell, D. W. (Indiv.) P. O. Box 2376, San Antonio, Texas
Chou, Wen-yen (St.) 2804 Whitis, Austin, Texas
Collier, William Thurmond (St.) 2422 Washington, Amarillo, Texas
Hartman, H. F. (Indiv.) Humble Oil & Refining Co., P. O. Box 3292, Baytown, Texas
McCarty, Sam T. (Indiv.) Beeville Concrete Products, P. O. Box 15, Beeville, Texas
Ossa O., Victor Manuel (St.) Box 1861, University Station, Austin 12, Texas
Prewit, John Wilkes (St.) 2310 Montana St., El Paso, Texas
Sundstrom, Alan Trexler (St.) 500 W. 8th St., Austin, Texas
Swenson, Stuart G. (St.) 325 S. Winnetka, Dallas 11, Texas
Taylor, Larry T. (Indiv.) 405 Bridge St., Weatherford, Texas

Utah

Brazier, Bernis E. (Indiv.) 412 Beneficial Life Bldg., Salt Lake City 1, Utah
Ferrell, William R. (St.) 1611 E. 3045 South, Salt Lake City 6, Utah
Mitchell, Donald Ralph (St.) Box D, College Hill, Logan, Utah
Olson, Linne M. (Jr.) P. O. Box 1307, Ogden, Utah

Virginia

Easterly, Harry W., Jr. (Indiv.) Box 1223, Richmond, Va.
Sadler, Joseph Bernard (St.) 1510 Hampton Blvd., Norfolk 7, Va.
Seaboard Air Line Railroad Co. (Corp.) 752 S. A. L. RR Bldg., Norfolk 10, Va. (W. N. Downey)

Washington

Milliken, Ralph R. (Indiv.) Box 254-B, Coulee City, Wash.
Phillips, Andrew Johan (Indiv.) 402 W. 5th St., Olympia, Wash.

Wisconsin

Foss, Richard L. (St.) 1456 Williamson St., Madison 4, Wis.
Korbitz, William E. (St.) Medford, Wis.
Paulson, James Marvin (St.) 608½ Fulton St., Wausau, Wis.

Wyoming

Adams, Richard S. (St.) 710 Bradley, Laramie, Wyo.
Barling, Lloyd Edward (St.) Meeteetse, Wyo.
Benes, Irvin J. (St.) 570 N. 9th St., Laramie, Wyo.
Boyd, Jack Welsey (St.) Box 150, Riverton, Wyo.
Brown, George Albert, Jr. (Jr.) 936 Summit St., Evanston, Wyo.

Brown, Ross P. (St.) 406 S. 9th St., Laramie, Wyo.
Covington, Dwight Archie (Jr.) 413 S. 6th, Laramie, Wyo.
DeBernardi, Alfred Valentino (Jr.) 1020 Lee St., Rock Springs, Wyo.
Donegon, Dallas Green (Jr.) 420 University Ave., Laramie, Wyo.
Kilgore, John Robert (St.) 211 Patch St., Laramie, Wyo.
Kurtz, James A. (St.) c/o N. V. Kurtz, Box 457, Sheridan, Wyo.
Liebl, John H. (St.) 212 S. 10th St., Laramie, Wyo.
Loss, Edward J. (St.) 150 E. "K" St., Casper, Wyo.
McBeath, Bernard C. (St.) Box 1006, Thermopolis, Wyo.
Rechard, Paul Albert (St.) 601 S. 13th St., Laramie, Wyo.
Satterfield, Claude R. (Indiv.) U. S. Bureau of Reclamation, Kortes Dam, Wyo.
Schmidt, John J. (St.) 143 N. Beech St., Casper, Wyo.
Seifried, R. D. (Jr.) 1317 Garfield, Laramie Wyo.
Stiles, William Homes (Jr.) Meeteetse, Wyo.

Guam

Maier, Karl J. (Indiv.) Station No. 10, Guam, Guam

Hawaii

Niwao, Jitsuo (Indiv.) 1388 Kilauea Ave., Hilo, Hawaii, T. H.

Puerto Rico

Grimes, James H. (Indiv.) Box 937, San Juan, Puerto Rico
Lrizarry, Samuel Diaz (Jr.) Victoria St., Lajas, Puerto Rico

Argentina

Franjetich, Zorislav (Indiv.) Calle Lavalle 1171, Buenos Aires, Argentina
Gorostiaga, Roberto (Indiv.) Parana 1255, Buenos Aires, Argentina

Australia

Bickford, Charles Reginald (Indiv.) c/o Maritime Service Board of New South Wales, Box 32, G. P. O., Sydney, N. S. W., Australia
Croudace, Gordon Morehead (Indiv.) CAM Premixed Concrete Engineering Pty., Ltd., James St. and Darley Rd., Leichhardt, Sydney, N. S. W., Australia
Hunt, Maurice (Indiv.) P. O. Box 340, Townsville, Queensland, Australia
Lovelock, O. I. (Indiv.) Hume Pipe Co. (Australia) Ltd., Upham St., Subiaco, Western Australia

Canada

Anderson, Harry Clyde (Indiv.) Dept. Public Works, Parliament Bldgs., Victoria, B. C., Canada
Bennet, Robert C. (Indiv.) 8 Bathurst St., Toronto, Ont., Canada
Dawe, B. E., (Indiv.) 9242 Hudson St., Vancouver, B. C., Canada
Douglass, Martin Keith (Indiv.) 2676 W. 42nd Ave., Vancouver, B. C., Canada
Francis, Ronald Lee, (St.) P. O. Box 377, Taber, Alberta, Canada
Holohan, E. J. (Indiv.) P. O. Box 38, Moncton, N. B., Canada
Hopkins, Lyle D. (Indiv.) Bridgewater, N. S., Canada
Lauer, Kenneth Robert (Jr.) c/o Hydro Electric Power Company, 8 Strachan Ave., Toronto, Canada
Sandwell, Percy Ritchie (Indiv.) 1204 Standard Bldg., Vancouver, B. C., Canada
Thorson, Victor (Indiv.) 1155 W. Pender St., Vancouver, B. C., Canada
Wooster, Hiram Ferris (Indiv.) 814 Birks Bldg., Vancouver, B. C., Canada

China

Chow, Li Hsiang (Indiv.) Yah Tai Consulting Engrs. & Archts., Room 308, 406 Kiangse Rd., Shanghai, China
Wang, Ming Yung (Indiv.) Yah Tai Consulting Civil Engrs. & Archts., Room 308, 406 Kiangse Rd., Shanghai, China

Costa Rica

Esquivel F., Hernan (St.) Av. 7. Cs. 4-6, Casa No. 448, San Jose, Costa Rica

Cuba

Lastra, Anselmo (Indiv.) San Nicolas 918, Havana, Cuba
Lopez, Julian Martinez (Indiv.) Linea 859, Vedado, Havana, Cuba

Denmark

Kampmann, Kierulff & Saxild S/A (Corp.) Dagmarhus, Copenhagen V, Denmark (J. Saxild)

Dominican Republic

Campos h., Manuel de Js. (Indiv.) Edificio Copello 410, Ciudad Trujillo, R. D.

Ecuador

Arroyo, Oswaldo (St.) Benalcazar 478, Quito, Ecuador, S. A.

England

Bowles, Reginald Ewart (Indiv.) Denmere, Telegraph Rd., Heswall, Cheshire, England

Evode Chemical Works Ltd. (Corp.) Glover St., Stafford, England

Grundy, Cecil Frederick (Indiv.) 6 North Court and Peter St., Westminister, London, S. W. 1, England

Martynowicz, Antoni (Indiv.) 3, Vaughan Ave., Samford Broa ., London W. 6, England

Peel, Edward Emery (Indiv.) 17, Preston Ave., North Shields, Northumberland, England

Sinclair, Alexander Stewart (Jr.) 28, Kenwood Rd., Stretford, Lancashire, England

Stagg, Rawson F. (Indiv.) "Sorgenfri," 68 Slayleigh Lane, Sheffield 10, England

Wolley, Charles Toole (Indiv.) c/o Messrs. Freeman, Fox & Partners, 68, Victoria St., London, S. W. 1, England

Honduras

Williams, Carlos (St.) Tegucigalpa, D. C., Republic of Honduras, C. A.

India

Archard, Herbert L. (Jr.) c/o Hind Constructions Ltd., 8 Royal Exchange Place, Calcutta, India

Bhandari, Ramnarainlal, (Indiv.) Ramkuti, Dhantoli, Nagpur 1, C. P., India

Darukhanawalla, J. M. (Indiv.) Prospect Chambers, 317-21 Hornby Rd., Fort, Bombay, India

Seshavatharam, C. (Indiv.) c/o Mr. Chegu Venkatappayya, Masulipatam, India

Italy

Ugo, Mantelli (Indiv.) Via San Bartolomeo degli Armeni 36, Genoa, Italy

Malta

Cemenstone Limited (Corp.) 61, South St., Valletta, Malta (Vincent A. Vella)

Mexico

Lozano, Enrique (St.) Ejercito Nacional 1°, Col. Nueva, Anzures, Mexico D. F., Mexico

Netherlands

Vries, J. De (Indiv.) Nieuwe Gracht 6, Utrecht, Netherlands

New Zealand

Whyte, D. O. (Indiv.) P. O. Box 1665, Wellington, New Zealand

Philippines

Bernales, Jose A. (Indiv.) 1160-R Magdalena St., Manila, Philippines

del Rosario, Ramon V. (Indiv.) 949 Santol, Sta., Mesa, Philippines

Perez, Jose F. (Indiv.) 970 Antonio Rivera St., Tondo, Manila, Philippines

Portugal

Vasconcellos, M. Paes de (Indiv.) Rua Jardim do Tabaco, 90-2° D°. Lisbon, Portugal

Scotland

Anderson, Frederick (Indiv.) 52 Falcon Ave., Edinburg, Scotland

Siam

Chu, I. C. (Indiv.) c/o Christiani & Nielsen (Thai) Ltd., Oriental Avenue, Bangkok, Siam

South Africa

Brown, Gresley Thomas (St.) 12 Gillatts Rd., West Ridge, Durban, Natal, S. Africa

Provincial Roads Engineer (Corp.) Cape Provincial Administration, Wale St., Cape Town, S. Africa (W.P.F. McLaren)

Switzerland

Kappeler, Henri (Jr.) Bankstrasse 39, Uster, Switzerland

Venezuela

Goa, Castor Segundo (Jr.) Jardines del Valle, Calle 16 No. 4, Caracas, Venezuela

Koe-Krompecher, Pedro L. (Indiv.) Qta Ana-Paula, Lado Aserradero, Caracas, Urb. Guaicaipuro, Caracas, Venezuela

Honor Roll

February 1 to July 31, 1948

Prof. Newlin D. Morgan, Jr., of the University of Wyoming heads the Honor Roll for the period ending July 31, with credit for 29 new members. Second with 15 credits is Professor Morgan's father, Prof. Newlin D. Morgan, of the University of Illinois. The Institute is appreciative of their splendid contribution to the growth of the ACI membership, in a family contest. Thanks also for the efforts of Prof. Raymond E. Davis, University of California and Thomas E. Stanton, California Division of Highways, who are largely responsible for the fact that California is now our leading membership state with a total of 441 members, as of July 31, 1948.

Newlin D. Morgan, Jr. (Wyo.).....29
Newlin D. Morgan (Ill.).....15
Raymond E. Davis (Calif.).....14½
Thomas E. Stanton (Calif.).....14
James A. McCarthy (Ind.).....5
Howard Simpson (Mass.).....5
Harrison F. Gonnerman (Ill.).....4½
Wm. H. Thoman (Colo.).....4½
Raphael Callejas H. (Guatemala).....4
E. I. Fiesenheiser (Ill.).....4
Henry L. Kennedy (Mass.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Elmo C. Higginson (Colo.).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
Phil M. Ferguson (Texas).....3
Robert W. Freeman (Calif.).....3
A. E. Mackney (Canada).....3
F. N. Menefee (Mich.).....3
Robert Morris (Colo.).....3
Richard A. Roberts (Calif.).....3
G. E. Troxell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
John C. Sprague (Ga.).....2½
Hugh Barnes (Calif.).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
Ray C. Giddings (Calif.).....2
Ernst Gruenwald (N. Y.).....2
Myle J. Holley, Jr. (Mass.).....2
Edward L. Howard, Jr. (Calif.).....2
John C. King (Ohio).....2
George E. Large (Ohio).....2
William E. Lumb (Canada).....2
Rene S. Pulido y Morales (Cuba).....2
Hugh E. Odor (Wash.).....2
James M. Polatty (Ga.).....2
Walter H. Price (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
John A. Ruhling (Ill.).....2
C. H. Scholer (Kans.).....2
E. Copeland Snelgrove (England).....2
E. W. Thorson (Colo.).....2
Joseph J. Waddell (Calif.).....2
Stanton Walker (D. C.).....2
Milton Fromer (Calif.).....1½
R. R. Kaufman (Ohio).....1½
Thomas C. Kavanagh (Pa.).....1½
Eugene Mirabelli (Mass.).....1½
Dean Peabody, Jr. (Mass.).....1½
Jerome M. Raphael (Colo.).....1½
Clarence Rawhouser (Colo.).....1½
Roy T. Sessums (La.).....1½
Harry F. Thomson (Mo.).....1½
John Tucker, Jr. (D. C.).....1½
Calvin T. Watts (La.).....1½
Harold E. Wessman (N. Y.).....1½
Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
A. Amirikian.....1
Boyd G. Anderson.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1
F. Barona.....1
Robert F. Blanks.....1
E. K. Borchard.....1
H. L. Bowman.....1
H. E. Burr.....1
John Butkus.....1
F. Campus.....1
Jose Luis Capacete.....1
A. Carbone.....1
Harold S. Carter.....1

(*Continued on p. 18*)

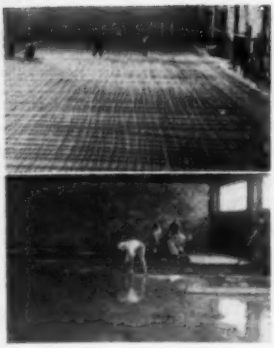
HONOR ROLL, *cont'd from p. 17*

C. O. Christy....1
Miles N. Clair....1
Rodger B. Collons....1
Jack E. Counts....1
Walter Dardel....1
J. Slater Davidson, Jr....1
Harmer E. Davis....1
A. N. Dutta Choudhury....1
George C. Ernst....1
A. Finlayson....1
T. H. Fleming....1
R. E. Fowlé....1
Frank M. Fucik....1
Carmelo Galindez....1
Anthony J. Giardina....1
H. J. Gilkey....1
R. R. Gore....1
Samuel Hobbs....1
W. C. Huntington....1
Phil C. Huntly....1
Wm. R. Johnson....1
H. D. Jolley....1
Edward F. Keniston....1
P. J. Kennedy, Jr....1
Orla A. Larsen....1
B. Leon....1
W. G. Limbach....1
L. D. Long....1
John V. Maescher....1
Clyde P. Mason....1
F. T. Mavis....1
William G. McFarland....1
Douglas McHenry....1
F. R. McMillan....1
Carl A. Menzel....1
Oliver H. Millikan....1
Leonard J. Mitchell....1
Robert B. B. Moorman....1
John J. Mullen....1
T. D. Mylrea....1
Amado B. Navoa....1
William T. Neelands....1
C H. Neill....1
D. G. Norman....1
Wm. D. Nowlin....1
W. E. Parker....1
R. A. Plumb....1
James J. Pollard....1
Horace A. Pratt....1
R. D. Rader....1
F. E. Richart....1
J. W. Rivett....1
Georges Ronai....1
E. Gonzalez Rubio....1
C. E. Sandstedt....1
Edward W. Scripture, Jr....1
Edgar A. Sealey....1
Thomas C. Shedd....1
Joseph Sill....1
Francis P. Sloan....1
Gustave Spirz....1
H. D. Sullivan....1
Hale Sutherland....1
E. O. Sweetser....1
T. Thorvaldson....1
Dean P. Tsagaris....1
Juan Agustin Valle....1
Walter C. Voss....1
Jukka E. Vuroinen....1
W. F. Way....1
Paul Weidlinger....1
J. M. Wells....1
C. D. Williams....1
Merle D Wilson....1

The following credits are, in each instance "50-50" with another member.

Robert W. Abbett	Manuel Castro Huerta
George C. Alden	Orville E. Jack
Oscar Vega Arguelles	Frank H. Jackson
A. Allan Bates	M. E. James
E. W. Bauman	Bruce Jameyson
C. T. Bishop	E. B. Johansen
A. J. Boase	Harry D. Jumper
Petrus Manzano Bonilla	Thomas M. Kelly
	John J. Kennedy Jr.
P. G. Bowie	K. J. Krawczyk
Harold W. Brewer	H. M. Larmour
A. R. Brickler	L. G. Lenhardt
Boyd S. Brooks	John F. Long
L. Rees Brooks	Wallace A. Marsh
Sterling Lowe Bugg	F. R. McCabe
B. W. Burns	Hubert F. McDonell
Jose Luis Casas	L. J. Messaros
Wilbur H. Chamberlin	Hugh Montgomery
Richard de Charms	Ray M. Moorhead
R. V. Chate	Israel Narrow
J. E. Christiansen	Elmer H. Oechsle
James H. H. Chun	Francisco Santos Oliva
H. W. Chutter	George L. Otterson
A. R. Collins	Chester B. Palmer
R. Torres Colondres	G. S. Paxson
F. Coppen	Donald O. Peck
Rolland Cravens	Ben Poisner
J. M. Crom	Chesley J. Posey
R. A. Crysler	Herman G. Protze
Charles W. De Groff	Warren Raeder
John G. Dempsey	Thomas J. Reading
J. E. Dunn	Dwight F. Roberts
J. Dvorak	H. H. Roberts
John N. Eckle	Bernard L. Robinson
Harlan H. Edwards	H. C. Ross
A. C. Eichenlaub	Fred H. Ryan
F. L. Ehasz	Anton Rydland
Arthur D. Engle	Herman Schorer
William E. Feidt	Joseph J. Shideler
Rudolph Fischl	Charles A. Shirk
E. M. Fleming	R. Simons
Leo V. Garrity	J. Singleton-Green
Carl V. Gezelius	Charles M. Spofford
J. R. G. Hanlon	A. R. Stuckey
Bernard Harrison	M. O. Sylliassen
Frank W. Haws	Ruth D. Terzaghi
H. L. Henson	J. Neils Thompson
A. E. Hiscox	A. G. Timms
W. C. Hodgkins	Donald R. Warren
John Hogan	J. C. Watt
G. W. Holcomb	J. D. Whittaker
H. N. Howe	William H. Wisniski
Walter L. Huber	Harry C. Witter
J. W. Hubler	Roderick B. Young

Better "ring" the dates on your calendar for the Institute's 45th Annual Convention, Hotel Pennsylvania, New York City, Feb. 22, 1945 (Board and Technical. Activities Committee only); Feb. 23 morning—Technical Committee meetings; afternoon first general session with other general sessions Feb. 24 and 25.



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AUTOLENE LUBRICANTS COMPANY
Industrial Division, Denver 9, Colorado

If you have something on your mind about concrete that might profitably occupy the minds of ACI members write "a letter to the editor." If it is something suitable for the new department "Letters from Readers," (p. 81) good! If it is something the Technical Activities Committee should look into—something for somebody else to do, it will be considered attentively. Address Editor, Journal of the American Concrete Institute, 717 New Center Bldg., Detroit 2, Michigan.

$500 CRSI award

A $500.00 cash award has been established by the Concrete Reinforcing Steel Institute "for outstanding contributions to the advancement of reinforced concrete construction." Any person, firm or corporation within the continental United States is eligible to compete for and receive the award as based on research contributing to the advancement of reinforced concrete, noteworthy contributions to the theory of reinforced concrete design, or improvements in reinforced concrete construction practice.

The initial award committee, appointed by the president of the Concrete Reinforcing Steel Institute, was named on the basis of one nomination each from the American Concrete Institute, American Society of Civil Engineers and Associated General Contractors of America. All three men are ACI members: Arthur J. Boase, Portland Cement Association; Frank E. Richart, University of Illinois; Henry C. Turner, Jr., president, Turner Construction Co. In the invitation to ACI, A.G.C.A. and A.S.C.E. there was indication of a desire on the part of the Concrete Reinforcing Steel Institute that the fields of interest of the three committee members be research, design theory and construction practice. Mr. Turner was named by the contractors' group, Professor Richart by the civil engineers and Mr. Boase by the American Concrete Institute.

One member of the committee is to serve, under the C.R.S.I. rules and provisions, for a 1-year term and be chairman in that year; one is to serve for 2 years, to be chairman in the second award year; one for 3 years, to be chairman the third year. Succeeding members of the award committee are to serve for 3-year terms, the chairmanship rotating each year.

The award committee is vested by C.R.S.I. with sole authority to select and name the recipient of the award, provided there is unanimous consent of all three committee members. Any person associated with a firm or corporation holding membership in C.R.S.I. shall not be eligible to serve as a member of the award committee. The committee is scheduled to meet annually and to make its decision within a 3-day period. If in the opinion of the award committee no outstanding contribution has been made to the advancement of reinforced concrete construction during any award year, no award shall be made for that year. The first award will be made at the C.R.S.I. 25th anniversary meeting at White Sulphur Springs, West Virginia, during the week of July 11, 1949.

C.R.S.I. makes no restricting stipulations as to the channels of publication through which the "outstanding contributions" shall have become known, and the award committee members are not required to devote any of their time to a search for award material—considering only such material as is brought to their attention by any interested person. C.R.S.I. reserves the right to withhold awards in any certain year or years, in which case the award committee is to be notified not later than 30 days after the beginning of any award year—May 1 to April 30.

William G. Grove

Word has been received of the death of William G. Grove May 27, 1948, in Lansing, Mich. An Institute member for 11 years, Mr. Grove was born in Philadelphia, Pa., November 28, 1882, and graduated from the University of Pennsylvania in 1909. His early experience was with the American Bridge Co. and Robison and Steinman, New York. During the war he was a special consulting engineer to the army engineer corps and was in charge of construction work at several large ordnance plants in the Mid-West, one of which employed the new type of poured concrete "igloo" ammunition dump for the storage of explosives. While with the War Department he shared credit for invention of the cable-stiffened truss.

After the war he became engineer of bridges and structures for the Connecticut

State Highway Department, and while in this position received an award from the American Institute of Steel Construction for his design of the Middletown-Portland bridge. Late in 1947 he retired because of poor health, and went to Lansing where he was living with his daughter, Mrs. T. P. Nanz at the time of his death.

William L. Shoemaker

William L. Shoemaker, junior member of the Institute died in Detroit, Mich. February 28, 1948. He was born in Seattle, Wash., May 12, 1921, and studied at Michigan State College where he graduated in 1943. He worked for a time with Lockheed Aircraft Corp. in Hollywood, was a radio technician in the navy, and later was employed by the Austin Co. in Detroit. At the time of his death Mr. Shoemaker was in the employ of R. W. Covey, Detroit consulting engineer.

By-laws and standards ratified

Institute members approved two new ACI standards and a change in Institute By-laws by letter ballot canvassed June 29, 1948. The standards are "Recommended Practice for Winter Concreting Methods" ACI 604-48, first published in the JOURNAL last December and reprinted again this month on p. 1, and the "Manual of Standard Practice for Detailing Reinforced Concrete Structures" ACI 315-48. This detailing manual of large format was published separately from the JOURNAL in 1946, and in view of the dwindling stocks will probably be reprinted without revision bearing the ACI Standard designation early next year. Revisions of Article II of the by-laws are as follows:

Change last sentence of Sec. 4 from "A year is here construed as the period between adjournment of two successive annual conventions", to "A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive conventions."

Change Sec. 6 from "The term of each officer shall begin at the close of the annual convention at which he is elected and shall continue until a successor is duly elected", to "The term of each officer shall begin immediately upon the announcement by the tellers of the canvass of ballots by which he is elected and shall continue until a successor is elected."

Change Sec. 7 from "A vacancy in the office of president shall be filled by the vice president having Institute membership seniority", to "A vacancy in the office of president shall be filled by the vice president having seniority in that office."

By-laws revisions are printed below.

Total number of ballots returned was 1137. The tally:

	Yes	No
Revisions of by-laws.....	1064	27
Adoption of standards:		
Winter concreting.....	1068	15
Detailing manual.....	1075	16

Members might clip these lines and insert p. 170 ACI Directory 1948

Sec. 4. The terms of office of the President and the Secretary-Treasurer shall be one year. At the first election after the ratification of these amended By-Laws, one Vice President shall be elected for a term of one year and one for a term of two years; thereafter one each year for a term of two years; at the first election after the ratification by these By-Laws, three Directors shall be elected for terms of one year, three for terms of two years and four for terms of three years; thereafter, four each year for terms of three years. A year is here construed as the period between the reports of Tellers on canvass of ballots for Board members at two successive annual conventions.

Sec. 6. The term of each officer shall begin immediately upon the announcement by the Tellers of the canvass of ballots which he is elected and shall continue until a successor is elected.

Sec. 7. A vacancy in the office of President shall be filled by the Vice President having seniority in that office

Motor trucks for ready-mix

"Correct Application of Motor Trucks to the Ready Mixed Concrete Industry," a paper presented by R. Cass at the January meeting of the National Ready Mixed Concrete Association, has been summarized in a recent technical information letter of that association. Mr. Cass outlines a procedure for the scientific selection of the proper truck unit for a particular operation, intended to eliminate much of the guesswork of the problem. By using long-established engineering principles and experience factors for different conditions of operation, requirements for truck units may be analyzed. Thus it is possible to select more accurately than in the past the particular truck chassis which will give highest operating efficiency and lowest cost in service. A detailed description of the procedure recommended for analyzing requirements of a truck unit is given in "Correct Application of Motor Trucks" published by Dartnell Corp., Chicago, Ill.

Construction first half 1948

Engineering News-Record for July 22, 1948, reports that: "Heavy engineering construction at the end of the first 6 months of 1948 is running 25 percent ahead of a year ago. Contracts awarded through June this year, as recorded by *Engineering News-Record*, total $3,333,-369,000 compared with $2,673,865,000 for half of 1947.

"Public contract awards for the first half of 1948 rose 51 percent above the 1947 6-months total while private contracts rose 5 percent. In the several classified construction groups listed below, public buildings showed the greatest gain rising to $509,416,000, or more than double the corresponding 1947 total of $252,419,000.

"All other groups, with the exception of unclassified construction, showed gains over 1947 ranging from 78 percent for bridges down to 6 percent for commercial buildings. Unclassified construction declined 38 percent from 1947 to 1948, but is still 26 percent above the corresponding 6-months total for 1946."

A.S.T.M. 1948 meeting

The American Society for Testing Materials held its 51st annual meeting June 21-25 in the Book-Cadillac Hotel, Detroit, Mich. with sessions including some 135 technical papers and reports and more than 360 meetings of its technical committees. Fifty-two new tentative specifications and tests were approved, and about 70 existing tentatives are to be adopted as standards subject to letter ballot of the society during the summer. Total registration at the meeting was more than 1775.

ACI member Theodore P. Dresser, Jr. of San Francisco was elected a director of the society. Other new officers include:

Richard L. Templin, New Kensington, Pa., president; L. J. Markwardt, Madison, Wis., vice president; and directors, Leslie C. Beard, Jr. and Simon Collier of New York, and Howard K. Nason and Edgar W. Fasig both of Dayton, Ohio.

Incidentally, the A.S.T.M. meeting brought welcome visitors to ACI offices.

Vicksburg to have new laboratory

Lt. Col. R. D. King, Director of the Waterways Experiment Station, Mississippi River Commission, Vicksburg, Miss., has announced award of a contract for the construction of an office and laboratory building to the Oden Construction Co., of Jackson, Miss., the low bidder. The new building, of concrete and masonry construction will provide office and laboratory space for a portion of the Concrete Research Division now located in temporary buildings.

The $256,000 contract calls for the furnishing of all plant, material and equipment (except that to be provided by the government) and performing all work for the construction of the building which will be located on the 800 acre reservation of the Waterways Experiment Station Sub-Office located near Clinton, Miss.

Tools, Materials, Services

Under this heading note will be made of producer literature having some content of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

A. C. HORN CO., 33-52 156th St., Flushing, N. Y.: pamphlet outlining uses, cost and handling techniques for Colorundum, a dry powder for dusting onto concrete surfaces as set begins, which, by special finishing and curing, is said to produce an abrasion-resistant colored finish. Another Horn publication records tests of a carbon black dispersion specially prepared for use with air-entraining cements.

THE PETER H. LOFTUS CORP., Pittsburgh: a 48-page booklet in observance of its 25th anniversary, entitled "A Quarter Century of Achievement." The publication describes Loftus engineering works, including steam power plants, coal mining operations, steel, aluminum, glass, food processing industries, government activities, consulting and machine design services.

BALDWIN LOCOMOTIVE WORKS, Philadelphia, Pa., sends "Stress-Strain Recorders," Bulletin 262, a 32-page catalog of Baldwin stress-strain recorders and strain followers. The bulletin describes and illustrates various types of Baldwin recorders; notes distinguishing features of 24 recorders for common makes of testing machines, and describes more than 50 extensometers, compressometers and deflectometers that can be used with the recorders. It shows typical stress-strain curves as produced by recorders, includes common testing accessories, and four types of strain gages.

SMITH CHEMICAL & COLOR CO., Brooklyn, N. Y., issues "Getting Results with Color in Concrete and Cement Products", 12 pages by Casper Smith, president of the company, discussing relative costs and quality of the pigments, notes their effect on concrete properties and comments on mixing of pigments and the blending of color into the concrete mix. He lists minerals used to obtain standard colors in concrete, citing differences between natural and synthetic coloring material; and warning against bad colors. Recommendations on quantity of color per bag of cement for terrazzo, cast and molded surfaces, troweled surfaces and mortar joints are noted.

BLAW-KNOX CO., Pittsburgh, announces production of a new portable bulk cement batching plant with provision for use of an auxiliary bin which the maker says permits easy and economical expansion of capacity. A long undertrack screw feeder, 20 ft 6 in. from hopper bottom to elevator, enables either immediate or eventual installation of a low-level auxiliary bin on the car unloading side of the elevator. An over-flow chute passes cement from the main or batching bin to the auxiliary bin, and the latter has a slide gate for passing material to the 9-in. screw feeder and thence by elevator to the main bin. The vertical elevator is of the enclosed bucket type rated at 50 tons per hour. Auxiliary and main bins are of the same design and are furnished either in 200-barrel or 400-barrel sizes, with total plant storage capacities of 200, 400, 600 or 800 bbls.

THE PERLITE MANUFACTURING CO., Carnegie, Pa., is now processing and distributing Perlite according to T. Coleman Ward, Jr., president. The manufacturers say that:

Raw perlite is a glasseous, volcanic rock, from the Rocky Mountain region. When pulverized, and heat-treated (or "popped") it becomes a white aggregate that weighs 3 to 12 lb per cu ft. Concrete block made from perlite aggregate weigh about half as much as standard block and have an insulating value of approximately 20 times as great.

In processing, the rock is crushed to proper size, preheated first to 900 F and then to 2000 F, thus vaporizing a small amount of water "in solution" in the rock. Steam so formed puffs up the softened material, and rapid cooling traps the bubbles within the perlite granules.

PUMICE PRODUCERS ASSOCIATION, was organized at the Biltmore Hotel in Los Angeles May 9 and 10 with T. Jack Foster, president; Leonard Finch, treasurer; and Lloyd A. Williamson as members of the board of directors. The organization proposes to conduct organized research, toward standardized specifications for pumice aggregate and to coordinate the efforts of individual producers. At the meeting, pumice users and producers discussed properties of pumice, its successful use in existing structures and its possible use in building units and precast products.

If you have something on your mind about concrete that might profitably occupy the minds of ACI members write "a letter to the editor." If it is something suitable for the new department "Letters from Readers," (p. 81) good! If it is something the Technical Activities Committee should look into—something for somebody else to do, it will be considered attentively. Address Editor, Journal of the American Concrete Institute, 717 New Center Bldg., Detroit 2, Michigan.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

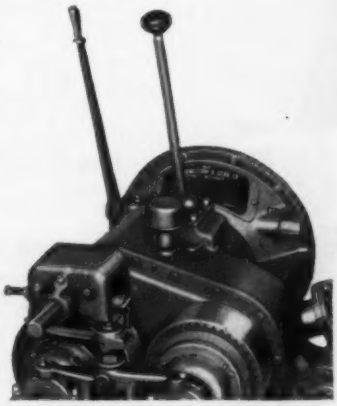
Available soon, for distribution to those who ask for it, will be a new 8-page pamphlet reprinting the synopses of all ACI papers and reports published in Journal V. 19 — Sept. 1947 to June 1948.

Those contents with concluding discussion and index to appear in the December 1948 Journal, Part II will conclude Proceedings V. 44. Just ask ACI, 717 New Center Building, Detroit 2, Michigan for the new synopses pamphlet.

PREVENTING SEGREGATION IN CONCRETE DURING TRANSIT

No additive insures against segregation of a concrete mix during transit, where length of haul, roughness of road, size and character of aggregates and slump of the mixture are all variable factors. Quality standards established by the ready-mixed concrete industry, and expected by users, make it imperative that no such segregation occur. The same exclusive features of Jaeger Truck Mixers or Truck Agitators which have made them the choice of the majority of producers of truck-mixed concrete, make them especially suited for delivery of any type of concrete, with or without an air entraining agent.

Automotive type transmissions afford operators of Jaeger Truck Mixers a choice of mixing drum speeds during transit — as low as 3 rpm for long hauls, agitation and slow discharge — as high as 12 rpm to insure thorough mixing on short hauls, and fast discharge where desired.



Two-Speed Transmission

Jaeger Top-Loading Sealed-Drum Truck Mixers or Truck Agitators have still another advantage in delivery of air-entrained mixes. Their air-tight mixing drum protects the batch from wet weather or drying and, in combination with the slow drum speeds mentioned above, assures accurate control of slump even on long hauls.

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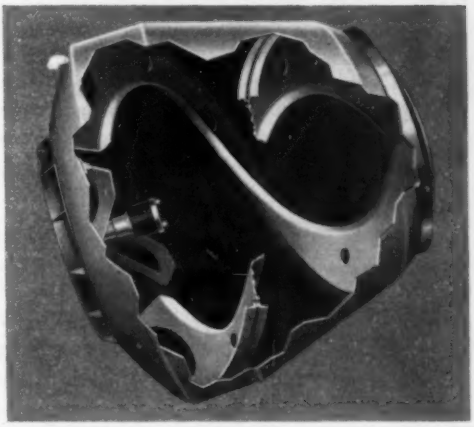

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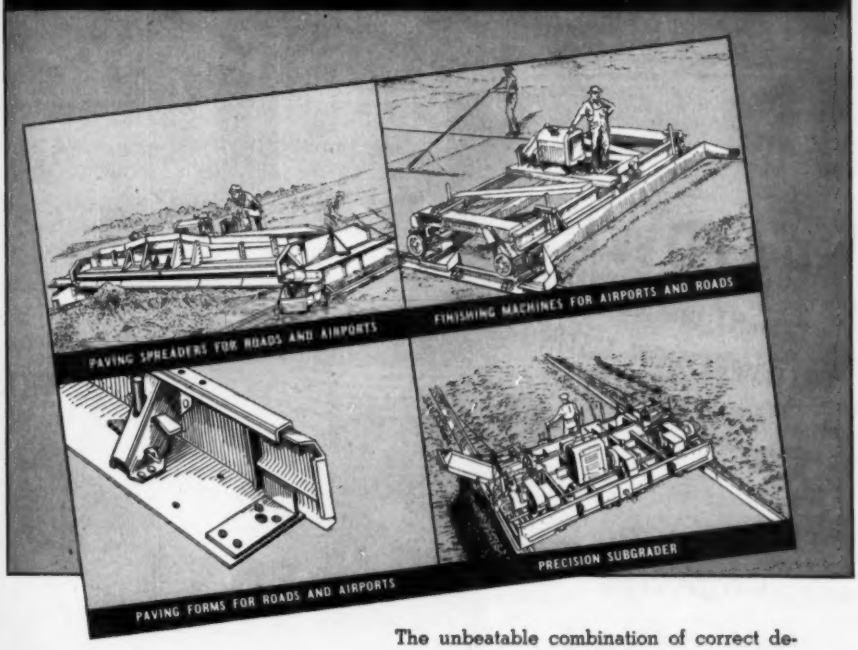
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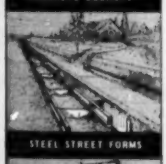

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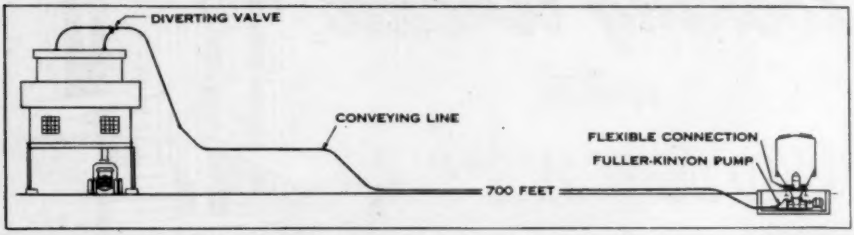

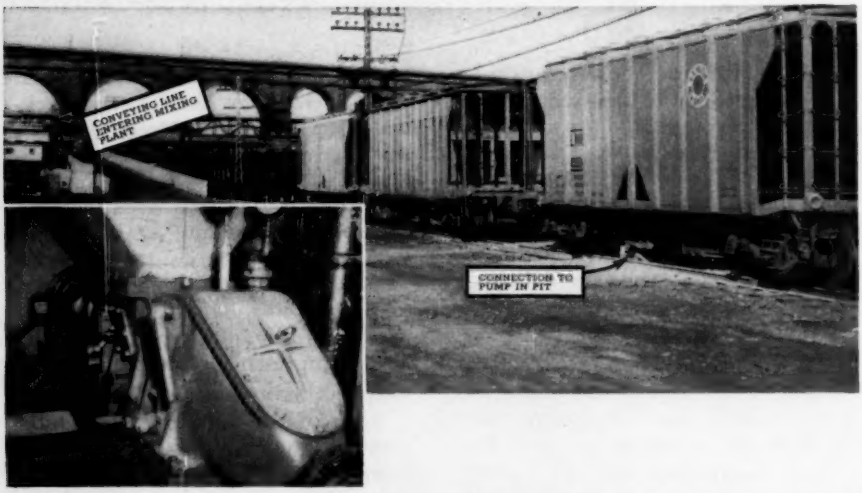

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Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention, and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy; $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

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DISCUSSION

Discussion closes January 1, 1949

Sept. Jl. '48

Recommended Practice for Winter Concreting Methods (ACI 604-48)—Report of Commitee 604

Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

While Professor Richart presents without recommendations a wholly objective research report of studies made at the University of Illinois—the first comprehensive re-investigation since Talbot's work 35 years ago—it is a reasonable conjecture that this paper portends a new basis for the design of reinforced concrete wall and column footings.—Editor

Reinforced Concrete Wall and Column Footings*

Part 1

By FRANK E. RICHART†

SYNOPSIS

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

*Received by the Institute July 14, 1948. Title No. 45-6a (in two parts, with No. 45-6b scheduled for November) is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 2, Oct. 1948, *Proceedings*, V. 45. Separate prints are available at 60 cents for each part. **Discussion** considered on the basis of the entire paper (copies in **triplicate**) should reach the Institute not later than March 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, Research Professor of Engineering Materials, University of Illinois, Urbana.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

INTRODUCTION

For many years, the methods used in the design of reinforced concrete footings have been based largely on the studies made on the subject by Prof. A. N. Talbot and published in 1913.[1] Since in recent years, many improvements have been made in construction materials and various changes have been made in design stresses and techniques, a re-survey of the subject has been needed to keep footing design abreast of technical progress. As a specific example of changes that have occurred since 1913, a major improvement in the quality of cement may be noted, with corresponding increase in concrete strengths. Marked improvements have been made recently in the effectiveness of deformations for reinforcing bars, particularly important because of prevalent high bond stresses in footings. There has also been a considerable tendency toward the use of stronger grades of steel, and some use of reinforcing units fabricated by use of welding. There has also been a gradual evolution in design methods.

The foregoing considerations led to an investigation of reinforced concrete footings, undertaken at the University of Illinois in 1944, in cooperation with the Committee on Reinforced Concrete Research of the American Iron and Steel Institute, and recently completed. The investigation involved tests of 24 wall footings and 132 column footings of fair size, made and cured in the laboratory and loaded in a testing machine utilizing a bed of steel car springs to simulate soil pressure.

The test footings were designed in general accordance with the methods given in the current ACI Building Code[2] and the 1940 Joint Committee Report[3]. In studying particular design features, however, such as bond stress, allowable stresses were naturally exceeded in some of the test pieces, in order to insure the type of failure desired and thus to establish limiting values of the stress under consideration. In the tests, extensive measurements of strains, deflections and slippage of bars were made to aid in comparing the results with the current design procedures.

Acknowledgment

Credit for initiating the investigation must be given to the A.I.S.I. Committee on Reinforced Concrete Research, and to the late R. R. Zipprodt, its research engineer, who gave especial attention to the conduct of the work. The planning of the tests was done by an advisory committee, consisting of the following: R. R. Zipprodt; F. E. Fahy, Bethlehem Steel Co.; E. E. Rippstein, Laclede Steel Co.; W. H. Thompson, Buffalo Steel Co.; and F. E. Richart.

Following the resignation of Mr. Thompson and the death of Mr. Zipprodt, the following were added to the committee: C. A. Willson, successor to Mr. Zipprodt; W. E. Bradbury, Sheffield Steel Corp.; A. P. Clark, American Iron and Steel Institute; and Svend Lassen, Truscon Steel Co.

Acknowledgment is also given to A. U. Theuer, E. J. Ward, C. E. Kesler and C. B. Clarke, research associates, who supervised the laboratory tests, to J. C. Ascherman, test assistant and to Eïvïnd Hognestad, research graduate assistant, all at the University of Illinois, who made important contributions to the tests or to the interpretation of the results.

Theoretical calculations

Moments—The moments considered here differ from the concept of moments originally assumed by Talbot. Referring to Fig. 1(a), by the usual definitions of mechanics, the bending moment on section A-A is the moment of all forces to the left of the section (while the moment about the centerline of the footing is theoretically slightly larger, the moment about A-A is simpler and its use is well justified). With a soil pressure, w, the bending moment M, about A-A, is

$$M = \frac{wc^2l}{2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

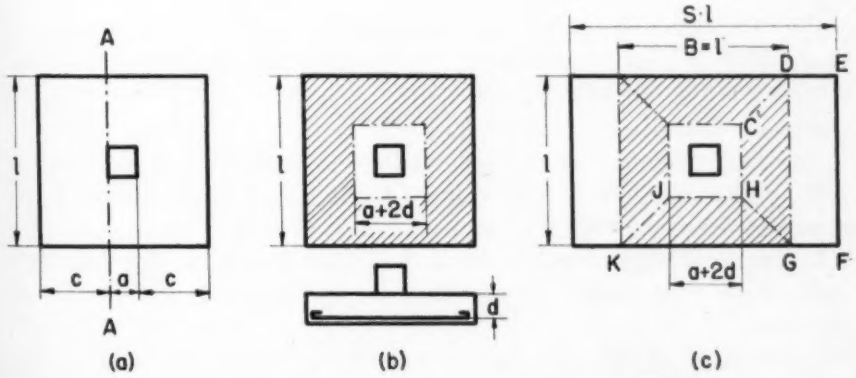

Fig. 1—Sketches showing design sections of footings

It should be noted that the ACI and Joint Committee design procedures both specify a design moment which is 85 percent of the moment given by Eq. (1).

Diagonal tension—The shearing stress, considered to be a measure of diagonal tension, is computed on a section at a distance d outside the limits of the column. The shear outside this section is given by the equation

$$V_1 = w \quad [l^2 - (a+2d)^2]$$

The shearing stress, computed on a width $b = 4(a + 2d)$, [see Fig. 1(b)] is as follows:

$$v = \frac{V_1}{bjd} = \frac{w\,[l^2 - (a + 2d)^2]}{4\,(a + 2d)\,jd} \qquad (2)$$

Bond—The maximum bond stress is computed on section A-A of Fig. 1(a), and is

$$u = \frac{V_2}{\Sigma ojd} = \frac{wcl}{\Sigma ojd} \qquad (3)$$

It will be noted again that the ACI and Joint Committee specify 85 pereent of the value given by Eq. (3), to be consistent with the equation used for maximum bending moment.

Rectangular footings—Two modifications of the above equations are specified by the ACI Code for rectangular footings. The bending moments in the two directions are taken at the face of the column but the reinforcement in the short direction is concentrated in the central part of the footing. When $S = 1\frac{1}{2}$, in Fig. 1(c), four-fifths of the short bars are placed in the central width B; when $S = 2$, two-thirds of the short bars are so placed.

For diagonal tension computations, the shear on the section CH of Fig. 1(c) is taken as the reaction on the area CDEFGH; the shear on JH is the reaction on the area GHJK.

Wall footings—The calculation of stresses in wall footings is assumed to follow the usual beam theory, except that the maximum diagonal tension is computed at a distance d from the face of the wall.

Outline of investigation

The investigation was divided into seven groups or series, one on wall footings and the rest on column footings. A brief outline of the seven groups is given in Table 1.

The wall footings of series 0 were 5 ft wide, 2 ft long, and carried a wall section 12 in. wide. Four plain concrete footings were included in the group.

The footings of series 1, 2 and 3 were all 7 ft square, carrying a short section of column 14 in. square. The three groups were designed to

TABLE 1—OUTLINE OF 156 FOOTING TESTS

Description of bars: 1—Atlantic, old style billet, deformed, intermediate grade.
2—Franklin, old style rail, deformed, rail steel.
3—Jones and Laughlin, new style billet, deformed, hard grade.
4—Laclede, new style rail, deformed, rail steel.
5—Laclede, plain bars, rail steel.

Series No.	Kind of footing	Intended manner of failure*	Design soil pressure, tons per sq ft	Kind of bars	No. of Tests
0	Wall	T	2, 3, 4	1, 4	24
1	Column	T, DT	1, 2, 3	3, 4	20
2	Column	DT, T	2	3	36
3	Column	B, T	2	1, 2, 3, 4, 5	56
4	Column	T, DT	2	3	4
5	Column, rectangular	T, DT	2	3	12
7	Column with capital	T, DT	2	3	4

*T indicates tension; DT indicates diagonal tension; B indicates bond.

fail primarily by tension, diagonal tension and bond, respectively. They constitute the major part of the investigation. Series 3 utilized five different types of reinforcing bar.

Series 4 contained four footings, comparable with series 3, but reinforced with welded bar mats.

Series 5 consisted of six rectangular footings 6 ft x 9 ft and six footings 5 ft x 10 ft long. All had a column stub, 14 in. square.

Series 6, originally planned on sloped and stepped footings, was omitted.

Series 7 consisted of footings comparable with series 2, but containing a capital 21 in. square, between the column and the footing slab.

MATERIALS, FABRICATION AND TEST METHODS

Materials

Cement—It was planned to use one brand of cement from a single mill throughout the tests. However, because of the long duration of the tests, and two periods of railway car shortages, another brand was substituted on two occasions. This contributed to a considerable variation in concrete strengths from the design values.

Fine and coarse aggregates—The fine aggregate was a Wabash River torpedo sand having an average fineness modulus of about 3.0. The coarse aggregate was a Wabash River gravel of 1-in. maximum size. Both are aggregates that have been in general use in the laboratory for years, and they passed the usual specification tests. Average sieve analyses are given in Table 2.

Concrete mixtures—The concrete mixture principally used was designed to have a 28-day compressive strength of 3000 to 3500 psi. In

TABLE 2—SIEVE ANALYSIS OF AGGREGATES

Average of 16 samples of sand, 20 of gravel

Kind of aggregate	Percentage of sample retained on sieve No.										Fineness modulus
	1 in.	¾ in.	½ in.	⅜ in.	4	8	16	30	50	100	
Sand					1	11	33	66	92	98	3.01
Gravel	2	24	54	81	96	98	98	98	99	100	6.94

TABLE 3—AVERAGE TENSILE PROPERTIES OF REINFORCING STEEL

Nominal sizes of bars were used in all computations.
All bars were round bars. The 1¼ square bar was an equivalent round.

Bar size, in.	No. of tests	Grade of steel	Yield point, psi	Ultimate tensile strength, psi	Percent elongation in 8 in.	Used in series No.
Atlantic, old deformed billet						
⅜	3	Inter.	52,900	78,600	19.5	0
⅝	3	Inter.	44,800	73,200	21.5	3
¾	6	Inter.	44,400	75,900	21.0	0, 3
Franklin, old deformed rail						
⅝	3	Rail	70,600	129,400	11.2	3
¾	3	Rail	66,500	115,600	12.2	3
Laclede, plain rail						
½	3	Rail	60,900	103,500	14.0	3
⅝	3	Rail	53,200	94,000	17.2	3
¾	3	Rail	73,000	117,900	12.9	3
Laclede, new deformed rail						
⅜	3	Rail	66,100	106,900	14.8	1, 3
½	3	Rail	74,700	115,600	13.4	1, 3
⅝	3	Rail	63,800	104,000	13.6	3
¾	5	Rail	69,200	137,000	9.5	0, 3
Jones and Laughlin, new deformed billet (first lot)						
⅜	3	Hard	66,800	98,100	17.5	1, 3
½ *	7	Hard	60,500	93,000	19.2	1, 2, 3
½	2	Hard	69,800	113,400	13.7	1, 2, 3
⅝ *	6	Hard	55,800	86,600	21.5	1, 3
⅝	5	Hard	64,600	102,900	16.1	1, 3
¾	3	Hard	52,700	97,400	16.7	1, 3
⅞	3	Hard	49,700	81,700	21.1	3
Jones and Laughlin, new deformed billet (second lot)						
⅜	11	Hard	76,900	111,300	15.0	1, 5
½ *	6	Hard	60,900	92,600	16.3	1, 5, 7
½	5	Hard	82,900	124,300	12.8	1, 7
⅝	5	Hard	61,600	104,600	15.8	1, 4, 5
¾	5	Hard	74,000	110,900	13.8	1, 4, 5
1	6	Hard	59,700	108,500	12.4	3
1¼	4	Hard	59,000	97,100	17.8	3

*Mixed lot containing two distinct qualities of steel.

TABLE 4—DIMENSIONS OF BAR DEFORMATIONS

Nominal areas and perimeters of bars are used. The bearing area considered is that normal to the axis of bar. The percent gap is the percent of the perimeter not offering a bearing surface.

Bar diam. in.	Description of bar	Average actual weight, lb per ft	Deformations				
			Average height, in.	Bearing area, sq in. per in.	Shearing area, sq in. per in.	Spacing per deformation, in.	Total percent gap
⅜	Atlantic, transverse deformation	0.376	0.049	0.037	0.80	0.76	32.2
⅝		1.047	0.062	0.090	1.50	0.95	23.6
¾		1.480	0.085	0.097	1.81	1.13	23.4
⅝	Franklin, longitudinal deformation	1.065	0.041	0.015	0.60	1.87	58.0
¾		1.480	0.051	0.013	0.84	2.19	63.6
⅜	Laclede, transverse deformation	0.388	0.021	0.081	1.12	0.29	5.2
½		0.670	0.025	0.089	1.47	0.41	6.4
⅝	Laclede, inclined deformation	1.038	0.022	0.045	1.36	0.50	30.6
¾		1.501	0.029	0.081	1.66	0.60	29.8
⅜	Jones and Laughlin inclined deformation 2nd lot	0.388	0.017	0.065	1.10	0.29	6.4
½		0.685	0.030	0.115	1.45	0.38	7.6
⅝		1.052	0.038	0.145	1.77	0.47	9.6
¾		1.467	0.038	0.142	2.11	0.56	10.6
1		2.649	0.042	0.164	2.86	0.73	9.0
1¼*		5.204	0.071	0.259	3.86	1.06	12.8

*Round bar, equivalent to 1¼-in. square bar.

series 2, two other mixtures, one leaner and one richer, were used. The three mixtures, by weight, were 1:4.1:5.8, 1:3.6:5.2 and 1:3.1:4.8, with corresponding water-cement ratios of 0.88, 0.79 and 0.71, by weight. The concrete as placed had an average slump of 5 in. The strengths of the control cylinders are listed with the results of the footing tests.

Reinforcing steel—The reinforcing steel was of five different types, obtained from four sources. Two types were the new style deformed bars furnished by the Jones and Laughlin Steel Corp. and the Laclede Steel Co. These particular bars were furnished to comply with the A.S.T.M. tentative specifications for bar deformations[4]. The others were old style deformed rail and billet steel bars, and the plain bar. Properties of the bars are given in Table 3. Profiles of the deformed bars were also measured, and significant information regarding the deformations is included in Table 4. Fig. 2 also shows the general appearance of the five types of bar, except that the Laclede deformed rail steel bars of ¾ and ½-in. diameter were of the transverse lug type, similar to the Atlantic bar shown.

Fabrication of footings

The footings were cast in steel forms, open at the top except for a

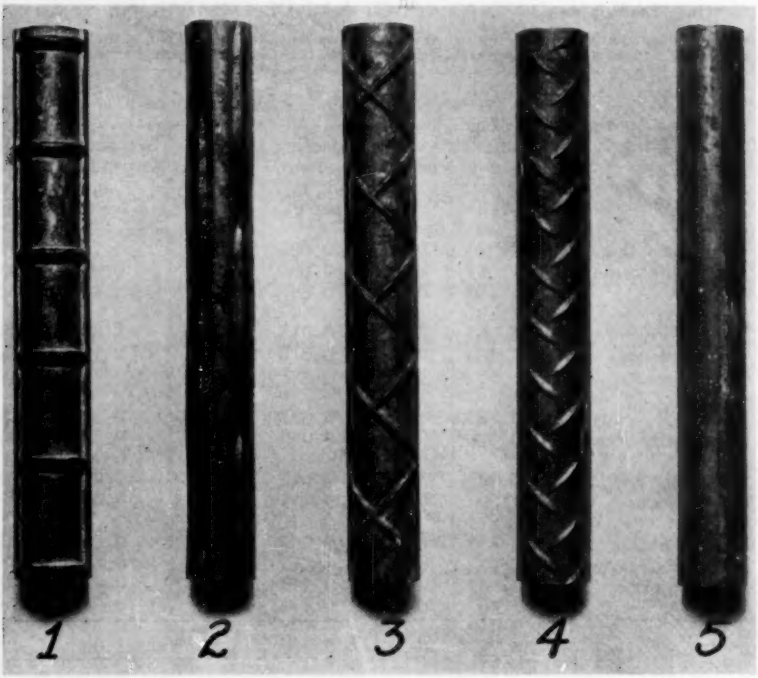

Fig. 2—Views of five types of reinforcing bars used. 1, Atlantic; 2, Franklin; 3, Jones and Laughlin; 4, Laclede, 5, Laclede plain

short suspended section, which formed the column stub. The reinforcing steel, wired together to form a mat, was placed at the proper level, and a section of electrician's loom was set in position for later use as a conduit for wires for electric strain gages. Surfaces of the reinforcing bars were ground smooth at sections where SR-4 electric gages were to be mounted later, and openings were provided in the concrete at these points by the insertion of tapered wooden blocks.

The concrete was placed in the form and compacted by use of a "Viber" high-speed vibrator. The footing and control cylinders were moist-cured under wet burlap for a week, then allowed to dry out for three weeks before testing.

In getting the footing ready for testing, the previously prepared sections of reinforcing bar were exposed and the SR-4 gages were attached and thoroughly dried out by use of a battery of heat lamps. Wire leads to all gages were inserted through the loom ready for attachment to the switching unit and portable strain indicator. Gage points were attached on the top of the footing for measurements of deflection and compressive strain.

In footings with straight reinforcing bars, a number of the bars were extended to the side surfaces of the footing, where slip of the bar with respect to the adjacent concrete was measured. In footings containing hooked bars, an extension of the straight portion was formed by welding on a short piece of ¼-in. bar, which reached to the side surface, where slip was measured as above.

Testing procedure

For convenience in conducting the footing tests, a special reinforced concrete base, mounted on four heavy wheels or casters, was constructed. This base, carrying the footing resting on car springs, was easily rolled in and out of the testing machine. With the footing centered in testing position in the machine, the wheels were removed and the base lowered to rest solidly on the platform of the machine. Fig. 3 shows a footing in position for testing in the Baldwin-Southwark testing machine.

With the electric gages all connected to the strain indicator, a small load was applied, the spherical bearing block at the top of the column stub was wedged to prevent further rotation, the load was released and a complete set of "zero" strain and deflection readings was taken. The deflection readings were taken on a gridwork of points on the top

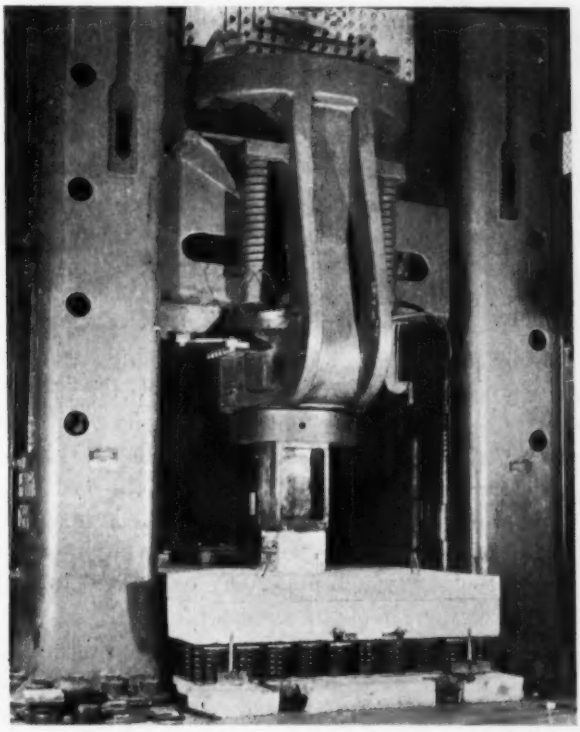

Fig. 3—View of a square footing in position for testing

surface of the footing, and as a check, readings were taken with a surveyor's level on the column stub and on the four corners of the footing slab.

Further loads were applied in about 15 increments until the footing failed, and a complete set of strains and deflections was measured at each increment. Notes were taken of the appearance of cracks on the edges of the footing and after the test the crack pattern on the bottom of the footing was marked with crayon and photographed.

TESTS OF WALL FOOTINGS

Description of tests

The footings of this series had a spread of 5 ft, projecting 2 ft on each side of a 12-in. wall. The sections tested were 2 ft long, had an effective depth of 8 in. and a total depth of 10 in. The reinforcement consisted of $\frac{3}{8}$ or $\frac{3}{4}$-in. round Atlantic bars of intermediate grade billet steel for footings 01 to 09 and $\frac{3}{4}$-in. Laclede deformed bars of rail steel for footing 10. Four footings were unreinforced. These four footings developed an average modulus of rupture at face of wall of 384 psi, or 10.5 percent of the compressive strength of the concrete.

The concrete used in the reinforced concrete footings had a compressive strength of 3970 psi.

The footings were tested in the 3,000,000-lb Baldwin-Southwark testing machine. The footings were supported on two layers of coiled car springs resting on the bed of the machine. Each layer consisted of 40 springs and the two layers provided a closure of spring of nearly 4 in.

However, even with this provision for differential deflection in the footing, the same difficulties were met in these tests as were encountered by Professor Talbot in his tests nearly 40 years ago.[1] The wall footing, subject to one-way flexure, is hard to test on a spring base, for the reason that when the yield point of the reinforcement is reached, cracks open rapidly in the slab opposite the wall faces and there is so much deflection of the projecting slabs that the distribution of spring pressure on the bottom of the footing can be determined only with difficulty. By comparing spring closures at different points, the variation in pressure may be estimated at different stages of the test, but with the distribution changing as the loading advanced, this became a laborious procedure. In several cases, the springs closed completely beneath the wall section of the footing, which made any calculated footing pressures meaningless thereafter.

The manner of failure of the wall footings indicates that such members behave like the ordinary flexural member. In the study of the results, attention will be paid to the "critical load" at which the yield stress of

the reinforcement was reached. Since the deflection of all footings at this stage was only about 0.1 in., it is evident that the distribution of the spring reactions was essentially uniform at this point.

Steel strains and deflections

Strains were measured on several bars at the design section opposite one or both faces of the wall. Average strains for each pair of footings are plotted against footing load in Fig. 4. The curves show a typical curvature near the lower end, indicating cracking of the concrete, and a curvature at the upper end, indicating yielding of the steel, at strains of 0.0016 to 0.0018. Here the yield point thus indicated agrees well with the value from tensile tests of the ⅜-in. bars, but is slightly higher for the ¾-in. bars, using the average value of modulus of elasticity of 29,000,000 psi found from the tensile tests.

Deflections of the ends of the footing with respect to the center of the wall have been plotted in Fig. 5, with each curve representing the average value for companion footings. The break in these curves, indicating yielding of the steel, agrees fairly closely with the "critical load" found from Fig. 4. From the data of these two figures, the critical loads given in Table 5 have been obtained. It is of interest that the calculated steel stresses for the critical load, given in Table 5, also agree very well with the yield stress for the steel used; such an agreement is expected only at high stresses in the steel.

Manner of failure

The important cracks in all wall footings occurred at the plane of the face of the wall, developing with increasing loads until they extended about three-quarters of the footing depth. When once the yield stress of the steel was reached, these cracks continued to open until the footing failed or the springs beneath the wall closed tight.

A few other unimportant cracks formed—tension cracks in the slab and below the middle of the wall. In the latter case however, measured strains were definitely smaller than those at the standard design section. For the footings with medium or high percentages of reinforcement, diagonal tension cracks formed at high loads, after the yield stress had been exceeded.

There was some evidence of slipping of reinforcing bars and fairly high bond stresses were developed as shown by Table 5, but there was no indication of primary failure in bond. Especially noteworthy was the bond stress of 970 psi developed in footing 10, made with Laclede rail steel bars. Slip measurements indicate that footings 9b, 10a, and 10b, all made with straight bars, developed appreciable slips at the critical load, and a secondary bond failure may have led to the large

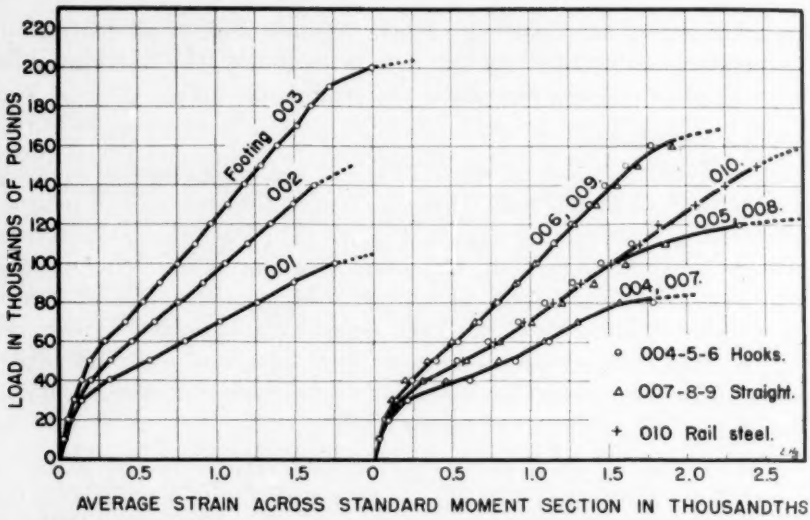

Fig. 4—Strain diagrams for wall footings (average of strains at design section, two companion footings)

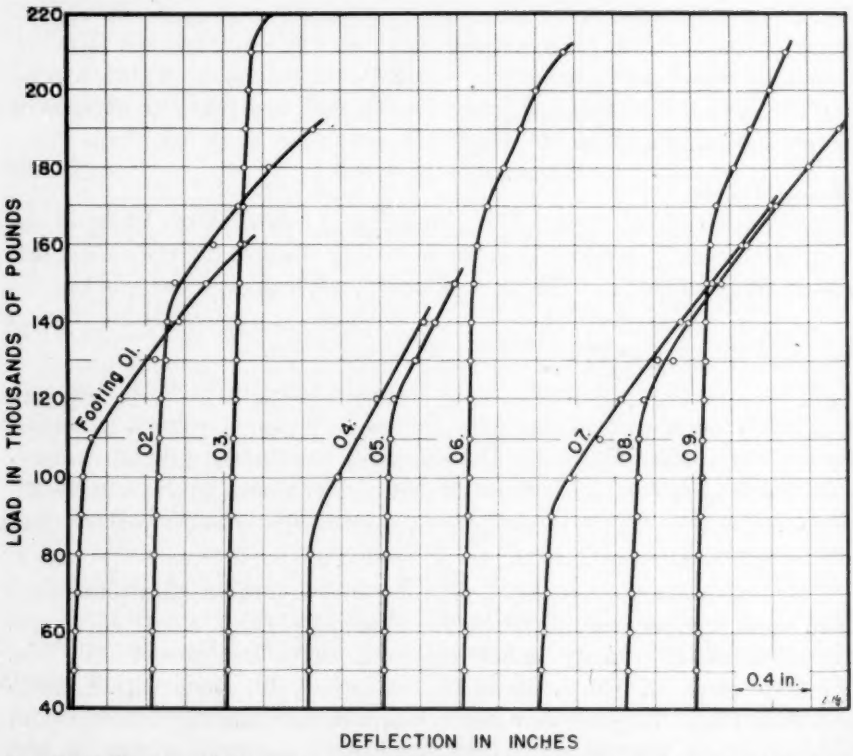

Fig. 5—Deflections of the center of wall footings with respect to the ends of footings (average for two companion footings)

TABLE 5—CRITICAL LOADS AND CALCULATED STRESSES IN WALL FOOTINGS

Reinforcement: Footings 1 to 3, ⅜ in. Atlantic, intermediate grade steel, yield stress, 52,900 psi.
Footings 4 to 9, ¾ in. Atlantic, intermediate grade steel, yield stress, 44,400 psi.
Footing 10, ¾ in. Laclede rail steel, yield stress, 69,200 psi.
Bars hooked at ends, footings 1—6, straight in footings 7—10.
All values represent average for two companion footings.
All footings 5 ft by 2 ft, 12 in. wall, 8 in. in effective depth.

Footing No.	Compr. strength of concrete f'_c, psi	Reinforcement		Load, kips		Calculated stress, psi at critical load			u/f'_c	v/f'_c
		Description	Area, sq in.	"Critical"	Est. max.	Tension f_s	Bond u	Shearing v		
1	3960	12–⅜ in.	1.32	105	158	53,000	412	169	0.104	0.043
2	4120	16–⅜ in.	1.76	147	185	55,700	433	237	0.105	0.057
3	4010	24–⅜ in.	2.64	215	230	54,300	423	345	0.105	0.086
4	3680	3–¾ in.	1.32	86	174	43,400	675	138	0.156	0.038
5	4120	4–¾ in.	1.76	120	173	45,500	706	193	0.171	0.047
6	3960	6–¾ in.	2.64	170	207	42,900	666	274	0.168	0.069
7	3825	3–¾ in.	1.32	90	162	45,500	708	145	0.185	0.038
8	4120	4–¾ in.	1.76	116	196	43,900	684	187	0.166	0.045
9	4120	6–¾ in.	2.64	170	221	42,900	666	274	0.162	0.066
10	3800	4–¾ in.	1.76	165	212	62,500	970	266	0.254	0.070
Av.	3970									

deflection of these footings, which produced closure of the springs beneath the wall section.

Estimated maximum loads

While the critical loads indicate the usable strength of a footing, the added load carried after the steel began to yield is of interest. Obviously, when the deflection of the footing became appreciable compared to the closure of the supporting springs, the distribution of the reaction became non-uniform. The load pattern can be estimated from the deflection measurements of the footing and the known properties of the springs. With the further observation that there was very little curvature of the projecting footing slabs, it may be assumed that the spring pressures varied linearly from wall to edge. The moment or shear is thus less than that based on an assumed uniform pressure. The reduction factor for moment or diagonal tension for footings of the dimensions tested can be shown to be $R_1 = \dfrac{D-2.3y}{D-0.4y}$, where D is the total closure of the springs at the middle of the footing and y is the difference in closure between the springs at the middle and those at the extreme edges of the footing. A similar reduction factor for bond stress is less affected by the deflection, being $R_2 = \dfrac{D-0.5y}{D-0.4y}$.

While deflection readings were not always taken to the point of failure, values of the maximum load, reduced by the factor R_1, are given in Table 5. While these values are approximate, they indicate an unexpectedly large margin of strength in these footings after yield stresses had been developed in the reinforcement.

Conclusions regarding wall footings

It may be concluded that all of the reinforced concrete wall footings failed initially by developing the yield stress in the steel, as demonstrated both by the strain measurements at the conventional design section, and by a sharp break in the load-deflection curves. The two agreed fairly well; where they did not, the latter criterion was chosen as giving the best evidence of yielding of all the bars at the design section. The critical load thus determined was further verified by the agreement between calculated stresses and the yield stress of the steel used.

Bond stress evidently was not important in the footings made with $\frac{3}{8}$ or $\frac{3}{4}$-in. hooked bars. At the critical load, there was no bond failure with the $\frac{3}{4}$-in. Atlantic straight bars, though there was appreciable slipping in footing 9b. These footings developed a bond stress of $0.17\ f'_c$ at the critical load. The bond stress in footings 10a and 10b reached $0.25\ f'_c$ at the critical load and there was considerable slipping at this load, though the footings developed about 25 percent more load before the supporting springs closed beneath the wall section.

No diagonal tension cracks developed until after the critical load was reached. Such cracks were noted in footings 3a, 3b, and 9b, which developed the highest computed diagonal tension stresses, 345 psi in footing 3 and 274 psi in footing 9. These stresses were 0.086 and 0.066 times the concrete strength, respectively.

Due to the non-uniformity of the spring pressures beyond the critical load, the estimated maximum loads deduced from the test results are not very reliable, but they do indicate an appreciable margin of strength beyond the load causing yielding of the steel.

TESTS OF COLUMN FOOTINGS, SERIES 1

Description of series 1

This series of tests was planned originally to furnish information on footings which should normally fail in the tensile reinforcement. To this end, three groups of footings were designed, assuming a concrete strength of 3000 psi, and with steel percentages of about 0.2, 0.4 and 0.6. Using hooked reinforcing bars, and an allowable steel stress of 20,000 psi, the designs used had less than the allowable bond and shearing stresses in the first two groups. The shearing stress was slightly above the allowable value for the third group. However, the use of hard

TABLE 6—PRINCIPAL RESULTS OF COLUMN FOOTING TESTS, SERIES 1

All footings 7 ft square, with 12 in. square column. Effective depth, 14 in., cover 2 in. All reinforcing bars were round and hooked at ends; all were Jones and Laughlin deformed, hard grade billet steel except in footings 109R and 110R, where Laclede deformed rail steel was used. Average compressive strength of concrete, 3330 psi. Tensile and bond stresses were computed from full statical moment, shearing stress at a distance d from column.

Footing No.	Compr. strength of concrete f'_c, psi	Reinforcement: Description, bars each direction	Reinforcement: Percent	Yield stress, psi	Max. load, kips	Maximum calculated stress, psi: Tension f_s	Bond u	Shearing v	Manner of failure†
101a	3350	21–3/8	0.20	76,900	380	95,200	508	135	T
b	3125	21–3/8	0.20	76,900	370	92,700	494	131	T
102a	3160	12–1/2	0.20	60,900 *	320	77,200	561	113	T
b	3550	12–1/2	0.20	82,900	340	82,000	596	120	T
103a	3165	8–5/8	0.21	61,600	340	79,300	715	120	T
b	3235	8–5/8	0.21	61,600	363	84,700	765	129	T
104a	3285	7–3/4	0.26	74,000	400	75,100	802	142	T
b	3210	7–3/4	0.26	74,000	360	67,600	722	127	T
105a	3430	42–3/8	0.39	66,800	524	65,600	351	185	T
b	2390	42–3/8	0.39	66,800	404	50,600	269	143	DT
106a	3750	23–1/2	0.39	60,500 *	525	66,000	481	186	T
b	3620	23–1/2	0.39	69,800	474	59,600	434	168	T
107a	3620	15–5/8	0.39	55,800 *	474	59,000	532	168	T
b	3410	15–5/8	0.39	64,600	425	52,900	476	150	DT
108a	4160	10–3/4	0.37	52,700	475	62,500	667	168	T
b	3275	10–3/4	0.37	52,700	450	59,200	632	159	T
109a	2965	60–3/8	0.56	76,900	538	47,200	252	190	DT
b	3080	60–3/8	0.56	76,900	460	40,300	215	163	DT
110a	3210	33–1/2	0.56	60,900 *	500	43,800	319	177	DT
b	2700	33–1/2	0.56	82,900	520	45,600	332	184	DT
111a	3010	24–5/8	0.63	61,600	480	37,300	336	170	DT
b	3400	24–5/8	0.63	61,600	573	44,600	402	203	DT
112a	3470	20–3/4	0.75	74,000	480	31,600	336	170	DT
b	2815	20–3/4	0.75	74,000	520	34,200	365	184	DT
109Ra	4070	60–3/8	0.56	66,100	575	50,400	269	204	DT
b	4180	60–3/8	0.56	66,100	550	48,200	258	195	DT
110Ra	3160	33–1/2	0.56	74,700	508	44,500	324	180	DT
b	3500	33–1/2	0.56	74,700	593	52,000	379	210	DT

*Two qualities of bars, mixed, in lot.
†T indicates tension; DT indicates diagonal tension.

grade steel, not contemplated originally, raised the factor of safety in tension, so that while initial tensile failures occurred in the first two groups, they definitely did not in the third one.

The footings were tested in the 3,000,000-lb Baldwin-Southwark testing machine, as indicated in Fig. 3. The footings were supported on a bed of evenly spaced car springs. As will be shown later, the deflections of the footing were small in comparison to the amount of closure of the car springs until the yield point of the reinforcement was reached, so that an essentially uniform support was provided over the bottom of the footing.

The principal results of this series of tests are presented in Table 6.

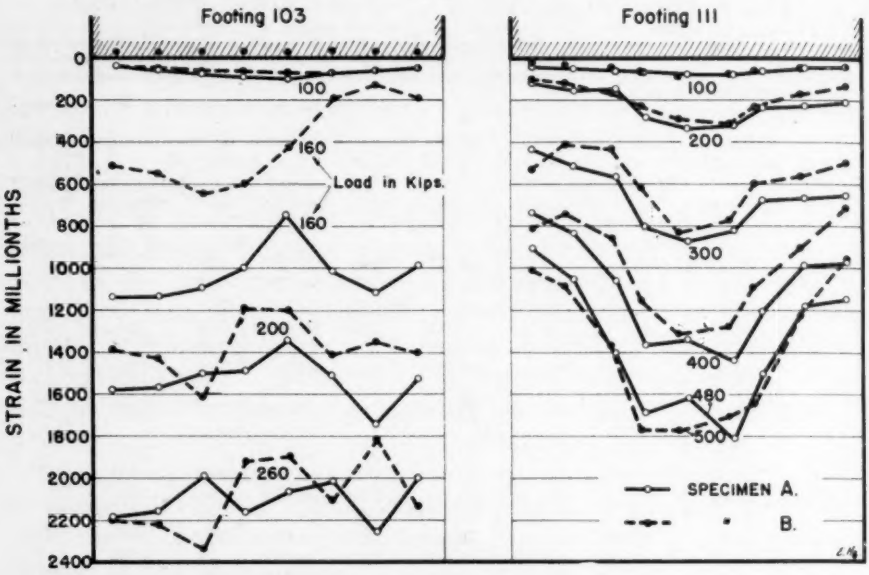

Fig. 6—Typical diagrams showing distribution of strains across standard design section, series 1

Strains and stresses at the standard design section

While the maximum moment theoretically should exist at the center line of the footing, strain measurements taken in a few of the tests showed no greater values than at the conventional section formed by a vertical plane in the face of the column, and justify the current use of this section for calculating design moments, as recommended by Talbot in 1913.[1] In the tests, strains were measured on six to ten bars at this section. Such strains were fairly small and uniform at loads which did not produce tension cracks in the concrete. With the formation of cracks, the strains became more erratic, depending on the location of cracks in the vicinity of the gage lines. As the yield point strain was reached the strains became even more erratic. Fig. 6 presents two typical diagrams showing the strain distribution across the footing. The first is for a lightly reinforced footing, the second for one rather heavily reinforced. In general, the strains were more nearly uniform in the first type; in the second they were generally considerably larger in the central part of the width, probably due to the greater variation in deflection in the footings carrying higher loads. The pattern of strain variation does not have great significance, so far as the load capacity of the footing is concerned, since, in tension failures, all of the bars undoubtedly reached the yield stress before failure occurred.

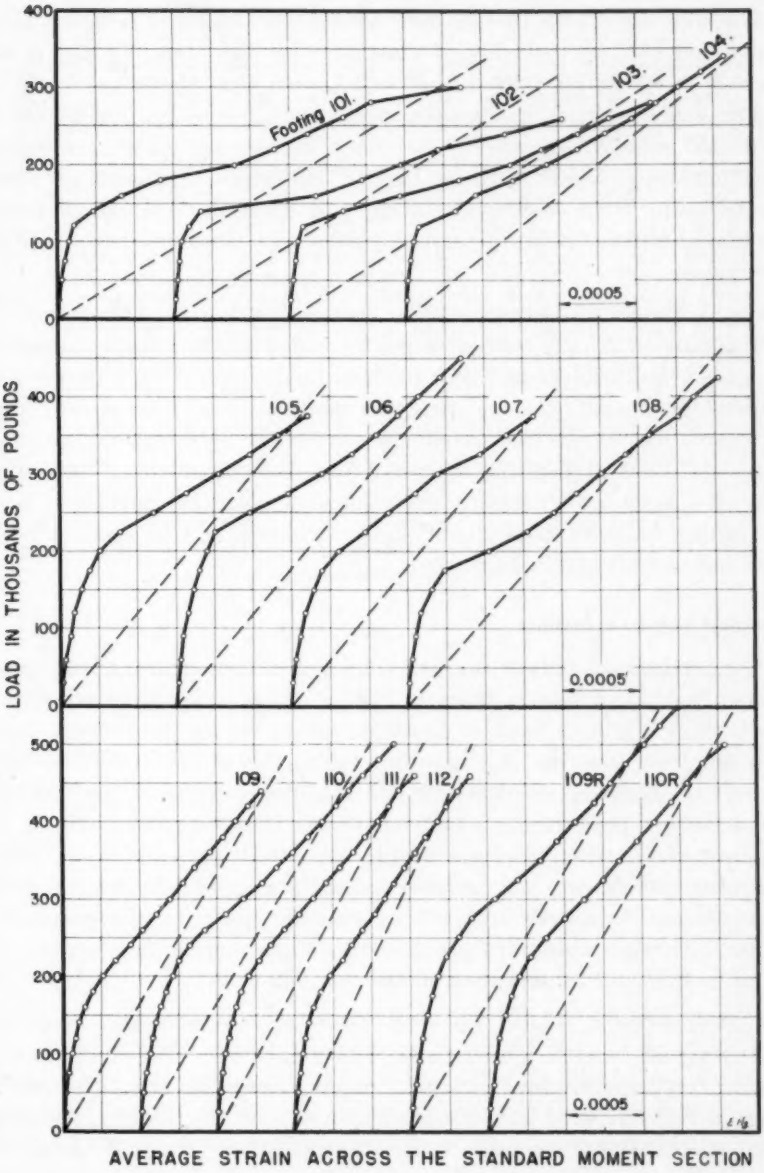

Fig. 7—Load-strain diagrams for footings of series 1 (average of strains across design section, two companion footings)

The average strain in the bars across the standard moment section, multiplied by the modulus of elasticity for the steel, should give the average stress in the bars. It is well known that the stress in the steel at low loads is small in comparison with stresses computed by the conventional straight-line theory, which presumes no tensile resistance in the concrete. After cracking begins, the stress increases at a much greater rate with increasing loads and generally the measured and calculated stresses converge at the yield stress, if no other type of failure intervenes.

Fig. 7 shows load-strain curves for the footings of this series. The strains plotted are the *average* values measured on the bars at the standard design section, and each curve represents the average for the two companion specimens. Also shown are straight lines representing strains corresponding to the calculated stresses on the standard design section, using the statical bending moment and the conventional straight-line flexural theory. The strains were derived from the calculated stresses by using a value of E of 29,000,000 psi, determined by tension tests of specimens of all of the bars used.

Maximum moment in footings

The load-strain curves of Fig. 7 furnish information on the bending moments that produce tension failure. Such information was not available in Talbot's tests of footings, which led to the present practice of using less than the full bending moments, as derived from statics, in design. Keeping in mind that the strains in excess of the yield point strain are not particularly significant, since they represent only a fairly constant yield stress, it is seen from Fig. 7 that, for tension failures, the measured strains near the maximum load correspond to the full statical moment, not to 85 percent of that moment, as currently assumed for design. There is abundant proof of this statement in the measurements taken in this and other series of tests to follow.

This conclusion would be even more apparent if the modulus of elasticity had been assumed to be 30,000,000 psi. However, from the tension tests made, and from the further knowledge that the weight of many of the deformed bars was slightly less than the nominal weight, the value of 29,000,000 psi used in interpreting these tests is believed to be well justified.

It is sometimes asserted that the low stresses in the steel at design loads justify a reduced design moment. However, this condition is equally true for beams and other flexural members. The maximum load is the best criterion of footing resistance, and it is on this load that the factor of safety of the structure is judged.

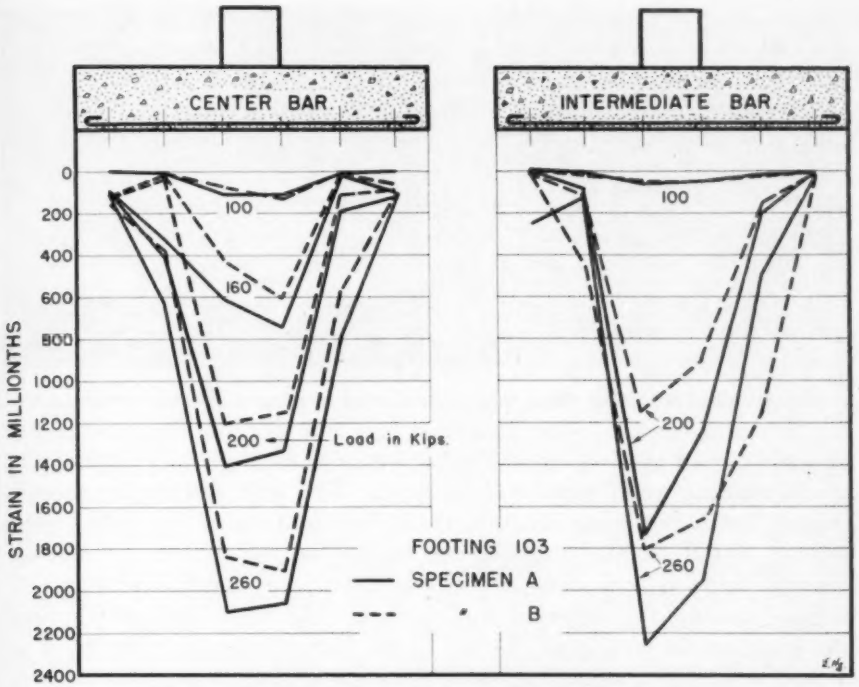

Fig. 8—Distribution of strain along reinforcing bars, series 1

Distribution of strains along a bar

Two typical diagrams showing the distribution of strain (and likewise the stress) along a bar, are shown in Fig. 8. The first applies to a bar near the middle of the footing, running beneath the column stub; the other is for a parallel bar, in the outer part of the footing slab, 16½ in. from the edge. At low loads, the curves are not particularly significant, but at high loads, the strains approach closely the parabolic shape corresponding to the diagram of bending moment.

That the shape of the strain diagram for the bar in the outer part of the slab is very similar to that for a center bar might have been inferred from the curves for strain across the standard design section, but these diagrams lend force to the conclusion that the entire width of the footing is effective in resisting bending. The slope of the strain curve, being proportional to the bond stress, also gives a good idea of the variation of bond stresses along the bar.

Shearing stresses

A feature of all of the tests of this series was that final failure of the footing was by diagonal tension, in which a section of concrete, which was a continuation of the column stub, flaring outward at about 45

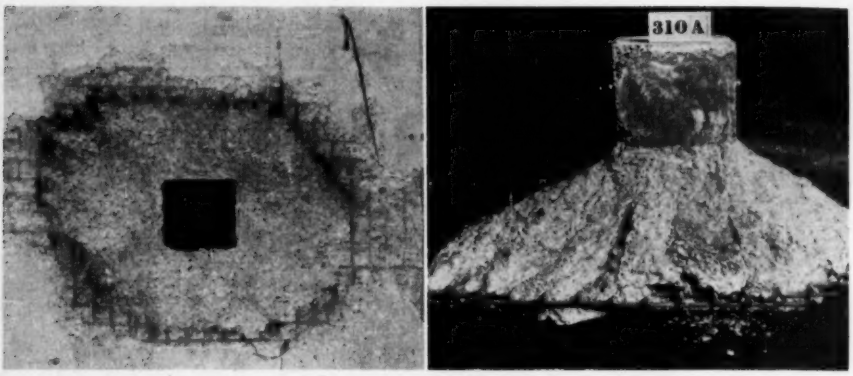

Fig. 9—View of footing after failure, with bars cut to allow separation of two parts

degrees toward the bottom of the footing, separated from the remainder of the footing and "punched" through. This was a diagonal tension failure, not a punching shear in the strict meaning of the term. The section of failure was undoubtedly formed at the bottom by tension cracks, which as they increased in size, took an inclination of about 45 degrees toward the edges of the column. The final shearing through the compression zone near the top of the footing may be thought of as a final shearing or punching, since diagonal tension was not present in that area.

Fig. 9 shows views of a footing after failure. The bottom reinforcing bars, which still tied the central pyramidal plug to the rest of the footing, were cut to allow the two parts to be separated for examination. The view of this footing, though from another series of the tests, is typical of the final shearing failure of these footings.

A glance at Table 6 shows that the results of the tests must be interpreted with judgment. The first group of eight lightly reinforced footings, which very obviously failed by tension, finally punched through at shearing stresses of 113 to 142 psi, computed on the conventional section at a distance d outside the column stub. This secondary failure at relatively low shearing stresses can only be explained by the fact that yielding of the steel produced very large cracks, which reduced to a minimum the section resisting shear and allowed the footing to collapse finally at abnormally low calculated shearing stresses.

Much the same reasoning applies to the second group of footings, with about 0.4 percent of reinforcement, though the shearing stresses varied from 143 to 186 psi. The shearing failures were again secondary, induced by the initial yielding of the tensile steel.

In footings 109 to 112, including the four made with rail steel, it is evident from both the measured steel strains and the stresses calculated

from the bending moments that no tension failures occurred. The bond stresses are also very low so that initial failure of these footings must be attributed to diagonal tension. The shearing stresses for the footings containing billet steel varied from 163 to 203 psi; for those containing rail steel, they varied from 180 to 210 psi. Expressed in terms of the compressive strength of the concrete used, these values range from 0.049 to 0.068 for the footings with billet steel and from 0.047 to 0.060 for those with rail steel. These results will be discussed more fully in the report of series 2.

Bond stresses

In most of the footings of the series the computed bond stresses were low. However, on four footings, types 103 and 104, the bond stresses, calculated by use of the full load, exceeded 700 psi, averaging 750 psi. In the first two cases, the steel yielded long before the maximum load was reached; in the latter two, it reached the yield point near the maximum load.

The bars used were all hooked at the ends, so that it is evident that complete failure by slipping could not occur. The measured slips displayed a characteristic of many of the tests. Some bars showed no slipping while others near by showed appreciable slip at high loads. Since end slip is a good indication that slip has developed along the whole length of the bar subject to shear, it is likely that such slip may be considered as contributing to the initial failure, along with the tensile yielding, particularly in the footings of type 104.

Summary

Some of the outstanding conclusions from series 1 may be repeated for emphasis:

1. The use of the conventional section for maximum moment appears well justified. While there is a marked variation in the stresses in bars crossing this section, the pattern of this variation is not consistent. It seems reasonable from the tests to consider all bars crossing the section as fully effective in resisting moment.

2. For footings failing initially in tension, the resisting moment derived from the measured strains agrees well at the load producing yielding of the steel with the full bending moment computed by statics; the use of 85 percent of this moment cannot be justified by these tests.

3. Footings which failed initially in tension, and had developed extensive cracking, finally failed in diagonal tension at relatively low shearing stresses. Such secondary failures cannot be considered typical of the resistance of the footing to diagonal tension. However, for footings which clearly failed by diagonal tension the range of shearing stresses, computed on the conventional section, extended from 0.047 to 0.068 f'_c.

TABLE 7—PRINCIPAL RESULTS OF COLUMN FOOTING TESTS, SERIES 2

All footings 7 ft square, with 12 in. square column. Jones and Laughlin improved deformed, hard grade reinforcing bars. Bars were evidently from two heats, with average yield stresses of 60,500 and 69,800 psi. Average values of f'_c, 2490 psi for footings 201 to 206; 3820 psi for footings 207 to 212; 4510 psi for footings 213 to 218. Calculated values of f_s and u are based on full statical moment. Values of v are based on section a distance d outside column. H = Hooked bars, S = Straight bars. All footings failed primarily in diagonal tension.

Footing No.	Compr. strength of concrete f'_c, psi	Reinforcement ½-in. bars each way		Eff. depth, in.	Max. load, kips	Maximum calculated stress, psi		
		No.	Percent			Tension f_s	Bond u	Shear v
201a	2660	42 H	1.00	10	298	28,700	209	204
b	2665				340	32,800	239	232
202a	2385	35 H	0.69	12	419	40,400	294	203
b	2180				400	38,600	281	194
203a	2630	30 H	0.51	14	420	40,500	294	149
b	2020				380	36,600	267	135
204a	2580	52 S	1.03	12	400	26,000	189	194
b	2540				400	26,000	189	194
205a	2260	44 S	0.75	14	460	30,200	220	163
b	2405				460	30,200	220	163
206a	2920	39 S	0.58	16	520	33,800	246	137
b	2680				610	39,600	288	161
207a	4250	42 H	1.25	8	360	43,400	315	364
b	4040				342	41,200	300	345
208a	4010	33 H	0.78	10	357	43,800	319	244
b	3845				380	46,600	339	260
209a	2890	27 H	0.54	12	440	55,000	400	213
b	2355				420	52,500	382	203
210a	4320	42 S	1.00	10	450	43,400	315	307
b	4230				400	38,600	281	273
211a	3660	35 S	0.68	12	480	46,300	336	233
b	4270				520	50,200	365	252
212a	3885	30 S	0.51	14	520	50,200	365	184
b	4100				500	48,200	352	177
213a	4500	42 H	1.25	8	340	41,000	298	343
b	4550				340	41,000	298	343
214a	4775	33 H	0.78	10	460	56,500	411	314
b	4940				479	58,800	428	327
215a	5045	27 H	0.54	12	480	60,000	436	233
b	4230				480	60,000	436	233
216a	4555	42 S	1.25	8	360	43,400	315	364
b	4470				360	43,400	315	364
217a	3720	33 S	0.78	10	380	46,600	339	260
b	4735				460	56,500	411	314
218a	4140	27 S	0.54	12	480	60,000	436	233
b	4430				440	55,000	400	213

TESTS OF COLUMN FOOTINGS, SERIES 2

Description of series 2

This group of tests was planned to furnish data on footings expected to fail by diagonal tension. Twelve footings were made with each of three grades of concrete; half of them were made with hooked bars, half with straight bars. All footings were designed by use of allowable stresses in tension and bond, but generally exceeding allowable shearing stresses. The Joint Committee Report[3] allows values of $v = 0.03f'_c$ for hooked bars and $v = 0.02f'_c$ for straight bars, while the ACI Code[2] specifies the use of hooked bars only and allows values of $v = .03f'_c$, but not to exceed 75 psi in any case. Referring particularly to the Joint Committee values, the test footings were designed with shearing stresses varying generally from the allowable values to more than twice the allowable values.

The footings were all 7 ft square, with effective depths varying from 8 to 16 in. The principal results of the series of tests are given in Table 7.

Stresses at failure

Before discussing the shearing stresses developed, it is well to consider the maximum tensile and bond stresses. The strain measurements show that in nearly all cases the tensile stresses were well below the yield stress of the steel. In footings 214, 215, 217 and 218, the stress evidently approached the yield stress and probably exceeded it in a part of the bars. The cracking of the concrete thus induced may have had a definite influence on the diagonal tension failure. As noted previously, when extensive cracking occurs after tension failure, the section resisting diagonal tension is undoubtedly decreased and the shearing stresses can be expected to be lower than when a primary shearing failure occurs.

The bond stresses were generally low and it is evident that no failure occurred primarily in bond.

All of the footings reached the maximum load when a pyramid of concrete beneath the column stub punched through suddenly and violently. All of the footings of this group may thus be classed as diagonal tension failures.

Shearing stresses and critical sections

The estimation of diagonal tension stress from the calculated shearing stress is established procedure in the theory of reinforced concrete. However, it is worth recalling that the method is an approximate one. Theoretically, in a homogeneous body in pure shear, the diagonal tension is equal to the shearing stress. When horizontal tensile stresses are added, the diagonal tension may greatly exceed the shearing stress. As shown by Talbot, the diagonal tension stress in a composite flexural

TABLE 8—MAXIMUM SHEARING STRESSES ON DIFFERENT CRITICAL SECTIONS, SERIES 2

The value x represents the distance from the column face to the critical section for shear; d is the effective depth of footing, and v is the shearing stress.

Footing No.	Compr. strength of concrete f'_c, psi	Eff. depth d, in.	Maximum load on footing, kips	Calculated stresses, psi, and ratios					
				$x = d$		$x = 2d/3$		$x = d/2$	
				v, psi	v/f'_c	v, psi	v/f'_c	v, psi	v/f'_c
201a	2660	10	298	204	0.077	271	0.102	316	0.119
b	2665		340	232	0.087	309	0.116	361	0.136
202a	2385	12	419	203	0.085	282	0.118	334	0.140
b	2180		400	194	0.089	269	0.123	322	0.148
203a	2630	14	420	149	0.057	216	0.082	265	0.101
b	2020		380	135	0.067	196	0.097	239	0.119
204a	2580	12	400	194	0.075	269	0.104	322	0.125
b	2540		400	194	0.076	269	0.106	322	0.127
205a	2260	14	460	163	0.072	237	0.105	290	0.129
b	2405		460	163	0.068	237	0.099	290	0.121
206a	2920	16	520	137	0.047	211	0.072	262	0.090
b	2680		610	161	0.060	247	0.092	307	0.115
207a	4250	8	360	364	0.086	462	0.109	530	0.125
b	4040		342	345	0.085	438	0.109	502	0.125
208a	4010	10	357	244	0.051	324	0.081	379	0.095
b	3845		380	260	0.068	345	0.090	403	0.105
209a	2890	12	440	213	0.074	295	0.102	355	0.123
b	2355		420	203	0.091	282	0.120	339	0.149
210a	4320	10	450	307	0.071	408	0.094	478	0.111
b	4230		400	273	0.065	363	0.086	424	0.100
211a	3660	12	480	233	0.064	322	0.088	387	0.106
b	4270		520	252	0.059	349	0.082	419	0.098
212a	3885	14	520	184	0.047	268	0.069	328	0.085
b	4100		500	177	0.043	258	0.063	315	0.077
213a	4500	8	340	343	0.076	435	0.097	500	0.111
b	4550		340	343	0.075	435	0.097	500	0.110
214a	4775	10	460	314	0.066	418	0.088	488	0.102
b	4940		479	327	0.066	435	0.088	508	0.103
215a	5045	12	480	233	0.046	322	0.064	387	0.077
b	4230		480	233	0.055	322	0.076	387	0.092
216a	4555	8	360	364	0.080	461	0.101	514	0.113
b	4470		360	364	0.081	461	0.103	514	0.115
217a	3720	10	380	260	0.070	345	0.093	403	0.109
b	4735		460	314	0.067	418	0.088	488	0.103
218a	4140	12	480	233	0.056	322	0.078	387	0.094
b	4430		440	213	0.048	296	0.067	355	0.080

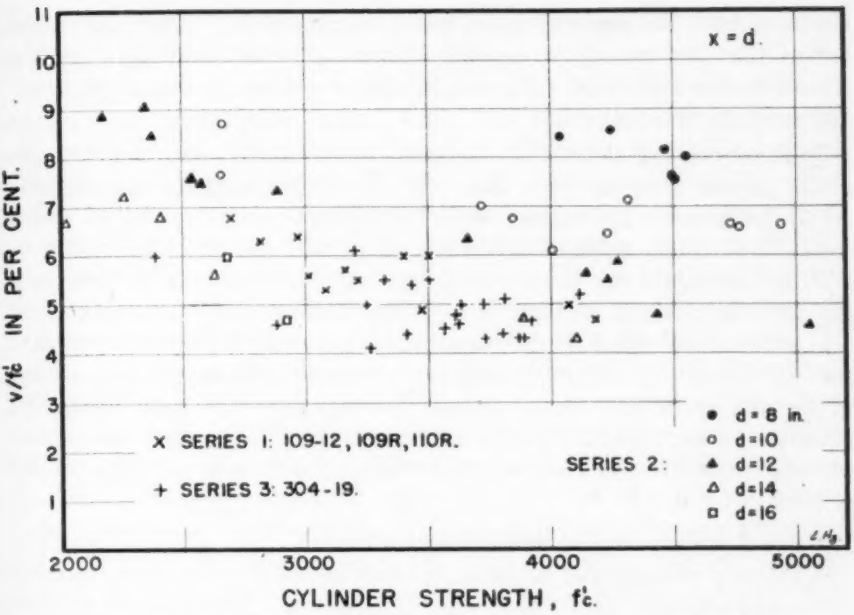

Fig. 10—Relation between values of v/f'_c and concrete strength f'_c

member, such as a reinforced concrete beam or slab, may frequently be two or more times as great as the shearing stress, depending on the relative values of tension and shear. Hence, the conventional use of shearing stress as a measure of diagonal tension resistance cannot be expected to apply uniformly to widely varying design conditions.

A further difficulty in the interpretation of footing tests is inherent in the fact that the value of the maximum computed shearing stress obviously depends upon the critical section considered. Professor Talbot, in his early tests, based his use of a shearing section a distance d (effective depth of footing) outside the column upon two principal factors: the observation that diagonal tension failures in footings produced fractures out and downward from the base of the column at an angle of about 45 degrees and, the fact that the shearing stresses computed on the section compared reasonably well with similar values obtained from beams and other flexural members. However, it is possible that some other section might be equally well justified. In Europe, for example, the distance $2d/3$ is used in some cases instead of d, and in the ACI Code, the distance $d/2$ is used for footings resting on piles. In slabs under concentrated loads, also, the shear at a distance $d/2$ outside the loaded area has been used in calculating shearing stress.

A study of the shearing stresses, based on critical sections taken at a distance d, $2/3\ d$ and $d/2$ is presented in Table 8. The table gives

values of both the shearing stress v and the ratio v/f'_c. While one might expect that the use of the smaller critical sections would produce less spread in the values of v/f'_c, the tabulated values do not show much difference in this respect.

It is interesting to note the relation between v/f'_c and the strength of the concrete, as shown in Fig. 10. The figure presents not only the results from series 2, but also those values from series 1 and 3 in which initial failure was apparently due to diagonal tension. The values of v/f'_c in this figure are those based on a critical section at a distance d outside the column. There is no very consistent variation between v/f'_c and the concrete strength, though the values are lowest in the central part of the figure, where medium concrete strengths apply. A number of the points in this area represent footings of series 3 for which the manner of failure was not clearly evident; some of the points are probably abnormally low because of cracks due to high steel stresses or slip of bars in bond.

Variation of maximum shearing stress with depth of footing

A rather striking result is obtained when the relation between shearing stress and effective depth of footing is examined. The following tabulation gives average values from series 2. (The distance from column to critical section is denoted as x.)

Effective depth, in.	8	10	12	14	16
Number of footings	6	10	12	6	2
v/f'_c when: $x = d$	0.081	0.070	0.068	0.059	0.053
$x = 2d/3$	0.103	0.093	0.094	0.086	0.082
$x = d/2$	0.116	0.108	0.123	0.105	0.102

This relation between maximum shearing stress and the depth of the footing was not foreseen when the tests were planned. In Talbot's tests in 1913, the ratio of effective depth to width of footing was 1/6 in all cases. In the present series, the ratio varies from 8/84, or 0.095 to 16/84, or 0.19. For a given load the shearing stress in a footing of 8-in. effective depth, and $x = d$, is nearly four times as great in as a footing of 16-in. effective depth. This is because three terms in the conventional equation, $v = V/bjd$, are affected by the depth; the value of V is greater for the thinner footing and the values of b and d are less.

In one respect the values in the foregoing tabulation are not strictly comparable. The footings of 8-in. depth and most of those of 10-in. depth, were made with the stronger grade of concrete while those of 16-in. depth and most of those with 14-in. depth were of the weaker concrete. While Fig. 10 would indicate that the concrete strength should

not greatly affect the value of v/f'_c, it is regrettable that footings of all depths were not made with all grades of concrete.

Considering the effect of high steel stresses and the accompanying cracks, it is found that footings 214, 215, 217 and 218, having the highest steel stresses at failure, gave values of v/f'_c generally below the average.

In trying to explain the higher shearing values for the thin footings, it was first conjectured that such footings might have developed enough deflection to make the spring reactions on the outer parts of the slab considerably less than in the central portion. If this were true it might make the actual shears less than those assumed in the calculations, with the discrepancy larger for thin footings than for thick ones. To pursue this possibility, the measured deflections of two thin footings, 207a and 213a, having 8-in. effective depths, were studied for a load near the maximum. The actual shear on the critical section, as calculated from the experimental deflection curves for these footings and the known stiffness properties of the springs, was found in both cases to be 1.6 percent less than the computed shear based on the assumption of a uniform reaction. This shows any that differences in the deflection of the several footings can cause only a negligible variation in the shearing strengths developed.

Another relation, which probably only adds confusion to the results from this series of tests, may be obtained from the data of Table 8. If the values of v/f'_c are compared for footings of the same effective depths, it is found that the values generally decrease with increasing values of f'_c. That is, the load capacity of such footings does not increase in proportion to concrete strength, as would be the case if v/f'_c were constant for all grades of concrete.

Effect of longitudinal reinforcement on shearing stress

In Table 7, it was noted that the thinnest slabs of each group had the highest percentages of reinforcement. This, of course, suggests a relation that has been known for many years but has never been recognized in design; namely, that the maximum shearing stress for beams without web reinforcement increases noticeably with the amount of reinforcement. This was shown by Talbot[5] in 1909 and more recently it was incorporated into a design formula, in a study of the behavior of welded stirrups, by Moretto[6]. He noted what seemed to be a dowel action of longitudinal reinforcement in beams. Such action contributed appreciably to the resistance of beams which failed essentially by diagonal tension. While the size of reinforcing bar probably should also be considered, Moretto assumed the contribution of the steel to be proportional to the percentage used.

When a footing fails by punching through of the column stub, it is reasonable that a part of the resistance is due to the local dowel action

of bars crossing the section of failure. This is a secondary element of strength, but it serves to explain the action of members in which a large local distortion of the steel occurs at the time of shearing failure. Moretto indicated that for the beams he tested the ultimate shearing stress, when no web reinforcement was used, could be expressed as

$$v = v_o + Sp$$

wherein v_o is the shearing stress corresponding to the resistance of the concrete to diagonal tension, S is a constant and p is the ratio of longitudinal reinforcement. Moretto suggested that the value of S for his test beams should be 5000 and this constant seemed to fit the results of his tests very well.

Applying Moretto's formula to these footing tests, and using a value of S equal to 10,000 since the steel ratio, p, is present in the footing in two directions, the effect of the consideration of the term Sp is to produce a much more nearly constant value of the shearing stress v_o.

Using the suggested procedure, and the location of the critical shearing section at a distance $2/3d$ from the column faces, values of the shearing stress v_o have been plotted in Fig. 11 against the concrete strength, f'_c. While the plotted points still show a considerable amount of scatter, they appear much more consistent than the similar points in Fig. 10. Values of shearing stress from series 1 and 3, used in Fig. 10, are also included here.

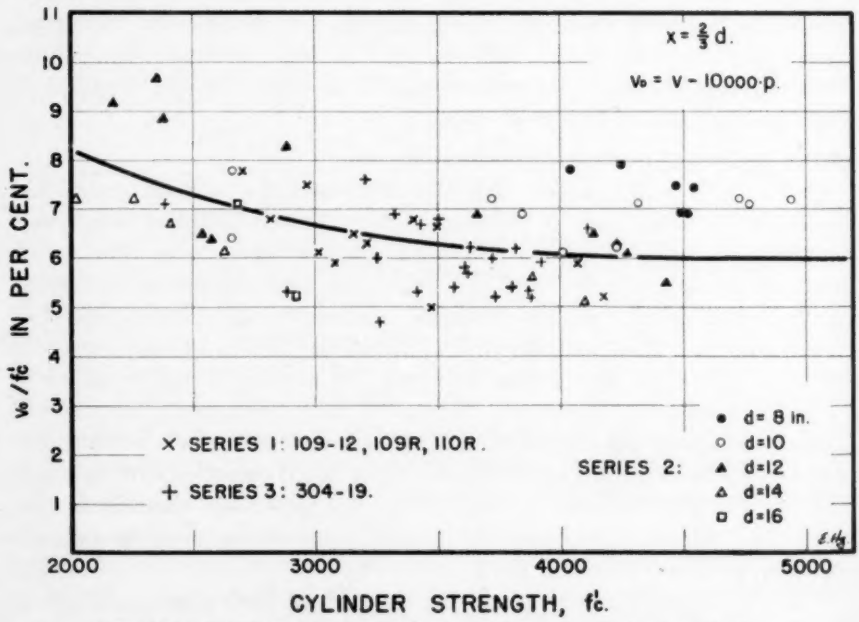

Fig. 11—Relation between values of v_o/f'_c and concrete strength f'_c

Regardless of whether or not a design formula should recognize the doweling effect of the main reinforcement, it must be admitted that the procedure used in Fig. 11 seems to reduce the variation of individual results from the average curve, as well as to suggest a rational and plausible reason for the scatter of test values computed by the conventional design procedure, as represented by Fig. 10.

Even when the effect of the reinforcing steel is compensated for, as in Fig. 11, the average value of v_o/f'_c is not constant for all values of concrete strength. The slight decrease in v_o with increase in f'_c is consistent with the variation between ordinary tensile and compressive strengths of concrete.

The procedure used in plotting Fig. 11 has been introduced as a means of explaining the observed test results, not necessarily as a design method. Obviously, the allowable stresses of an authority such as the ACI Code were selected to be used with the procedures specified in the code. If other procedures are to be used, the allowable stresses will need to be adjusted accordingly.

Effect of hooked and straight bars on shearing stresses

The reason for allowing a higher design shearing stress when hooked bars are used is that it is believed that many so-called shearing failures have been bond failures in disguise. If sufficient slipping occurs to permit extensive cracking of the concrete, the effect becomes similar to that produced by gross yielding of the steel; the section resisting diagonal tension is reduced and weakened, and the load required to produce a diagonal tension failure is thereby lowered.

The results of this series of tests seem to indicate that relatively little effect on shearing strength was produced by hooks on the reinforcing bars, which were all Jones and Laughlin improved deformed bars. Consider the values of v/f'_c for $x = d$ from Table 8. Average values for the three grades of concrete are as follows:

Concrete strength, psi	2490	3820	4510	Average
v/f'_c: with hooks	0.077	0.077	0.064	0.073
no hooks	0.066	0.058	0.067	0.064

The comparison afforded by this tabulation is not striking. Although there seems to be some beneficial effect of the hooks in the first two groups, the reverse is true in the last one. The average value for all hooked bar footings is 0.073 as compared to 0.064 for those with straight bars. In general the footings with hooked bars were thinner than the others.

From the magnitude of the bond stresses in Table 7, it may be concluded that bond was not an important factor in these footings at the loads at which shear failures occurred. Hooks cannot be expected to become effective as anchorage until some appreciable slip has taken place, starting at the critical section and proceeding toward the end of the bar. Had the bond stresses on the straight bars been excessive it seems likely that the effect of the hooks would have become much more evident.

The foregoing comments concerning bond stresses are substantiated by strain measurements, which show little difference in the stresses for straight and hooked bars. Thus, for footings 213 to 218, in which corresponding steel areas and footing depths are used, the load-strain curves at the standard moment section are very nearly alike for the two conditions of anchorage.

Factor of safety against diagonal tension

Since all footings of series 2 appear to have failed primarily due to diagonal tension, is is desirable to compare the values of the maximum

TABLE 9—FACTOR OF SAFETY AGAINST DIAGONAL TENSION, SERIES 2

Comparison of maximum shearing stresses with allowable values of v, from 1947 ACI. Building Code and 1940 Joint Committee Report. Critical section at distance d from column face.
Values shown are averages for two companion specimens.

Footing No.	Compr. strength of concrete f'_c, psi	Test values v, psi	Bar ends	Allowable values of v, psi		Factor of safety	
				ACI Code	Joint Committee Report	ACI Code	Joint Committee Report
201	2660	218	Hooked	75	80	2.91	2.73
202	2280	198	Hooked	68	68	2.91	2.91
203	2325	142	Hooked	70	70	2.03	2.03
204	2560	194	Straight	—	51	——	3.80
205	2330	163	Straight	—	47	——	3.47
206	2750	149	Straight	—	55	——	2.70
Ave.	2490						
207	4145	354	Hooked	75	124	4.72	2.64
208	3930	252	Hooked	75	118	3.35	2.14
209	2620	208	Hooked	75	79	2.77	2.63
210	4275	290	Straight	—	86	——	3.37
211	3965	242	Straight	—	79	——	3.06
212	3990	180	Straight	—	80	——	2.25
Ave.	3820						
213	4525	343	Hooked	75	136	4.57	2.52
214	4860	320	Hooked	75	146	4.26	2.19
215	4640	233	Hooked	75	139	3.10	1.67
216	4510	364	Straight	—	90	——	4.04
217	4230	287	Straight	—	85	——	3.37
218	4285	223	Straight	—	86	——	2.59
Ave.	4510						

shearing stress, v, with allowable values used in design. The ratio of the two stresses may be considered the factor of safety against this particular type of failure.

Table 9 presents a comparison of average test values for series 2 and allowable values for the grade of concrete used, computed in accordance with the 1947 ACI Building Code and the 1940 Joint Committee Report. The ACI Code does not permit the use of straight bars, but comparisons are made for hooked bars. In all but one case, the factor of safety is above 2.0 and for straight bars, for which the allowable value of v is $0.02\ f'_c$, the factor averages more than 3.0. For hooked bars, the average factor is 3.4 by the ACI Code and about 2.4 by the Joint Committee Report. The large difference in values is because of the ACI limitation of the value of v to 75 psi.

Summary

A few general conclusions may be drawn from the tests of series 2:

1. All of the footings of this group failed by diagonal tension, with a sudden, violent punching failure.

2. The maximum shearing stresses, computed on the conventional critical section varied considerably with the effective depth of the footing, being larger for the thinner footings.

3. The use of a critical section at less than the distance d outside the column faces seems worth considering in interpreting the test results, as does also the allowance of a portion of the maximum shear for the doweling effect of the reinforcing bars. These features are considered here as possible explanations of footing action, not at this time as suggested design methods.

4. For this series of tests, employing improved Jones and Laughlin deformed bars and with no high bond stresses present, the shearing stresses in footings with hooked bars were not particularly greater than those in footings with straight bars. The advantage of about 14 percent in average shearing strength, shown by the hooked bars may be partly due to other causes, such as differences in footing thickness, variations in concrete strength and similar factors.

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Lattice Analogy in Concrete Design*

By DOUGLAS McHENRY†

SYNOPSIS

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

INTRODUCTION

The lattice analogy for the solution of two-dimensional stress problems was devised with the particular needs of the concrete structural designer in mind. It will be recalled that the basis of most currently used design methods was laid in the early and middle 1700's when Bernoulli and Euler adopted the assumption of linear distribution of stress and developed the theorem of the proportionality of curvature to bending moment. It will also be recalled that concrete came into prominence as a structural material during the years 1900 to 1910. In other words, basic design concepts antedate the construction material now in use by about a century and a half. It is not surprising, then, that this newer material is being used to build structures—particularly, massive structures—to which the old design concepts are not well adapted. The lattice analogy is presented as a step toward the development of design methods which are more accurate for the particular type of structures for which concrete is used so extensively: massive structures, continuous

*Presented in brief at the ACI 42nd annual convention, Buffalo, New York, Feb. 19, 1946. Manuscript received by the Institute Feb. 23, 1948. Title No. 45-7 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 2, October 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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heavy frames, reinforced or composite structures, etc. It will resolve only a limited number of the designer's difficulties, for (at least in its present development) it does not consider the imperfect elasticity of concrete, and it is a laborious process. However, it is a simple method involving no difficult mathematics; it will handle two-dimensional sections of any shape and subjected to any sort of loading; volume changes due to temperature change or drying shrinkage may be introduced; cracks or joints may be taken into account, and reinforcement may be introduced.

APPLICATION OF THE LATTICE ANALOGY

The theory of the lattice analogy has been presented elsewhere* and will not be repeated in detail here. Stepping immediately into a simple practical application of the method, consider a deep beam loaded at the third-point, as shown by Fig. 1(a). With a span: depth ratio of 3:1,

*McHenry, Douglas, "A Lattice Analogy for the Solution of Stress Problems," *Journal* Institute of Civil Engineers, Dec. 1943, pp. 59-82.

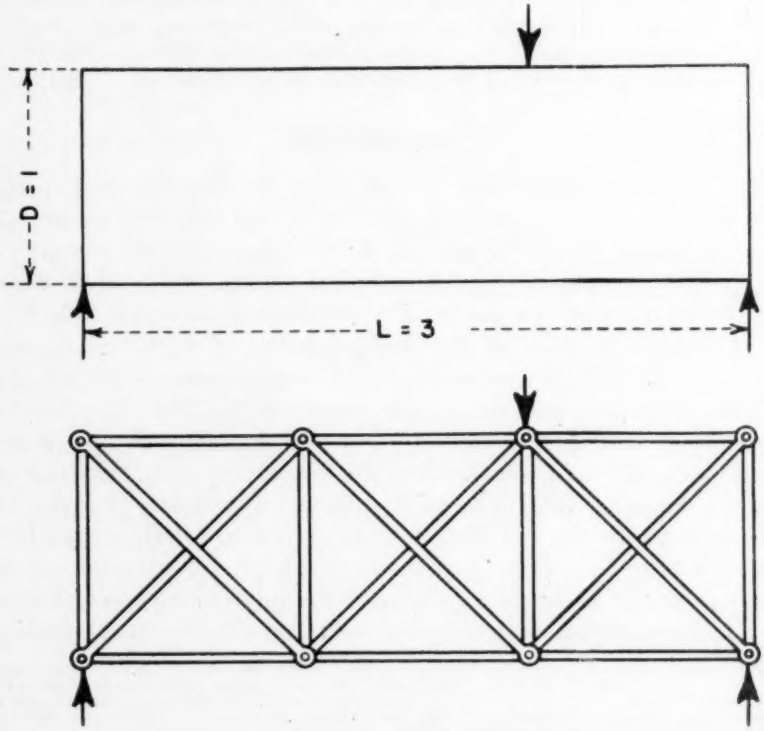


Fig. 1a (top)—Deep beam with unsymmetrical load

Fig. 1b (bottom)—Corresponding lattice of cross-braced square frames

we know that the computation of deflections and stresses based on the elementary theory of straight-line distribution will be more or less inaccurate. If the beam is a large and important member, the stresses may be determined by photoelasticity or by other model tests, or the methods of the theory of elasticity may be called upon. The difficulties and the limitations of these methods, one experimental and the other highly mathematical, are well known. Application of the lattice analogy is described in the following paragraphs.

First, the beam is simulated by a simple two-dimensional framework or lattice, made up of cross-braced pin-connected square frames, as shown by Fig. 1(b). The constants AE (area $\times$ elastic modulus) for the lattice members bear the following simple relationship to the modulus of elasticity of the prototype material:

For the horizontal and vertical members

$$AE_N = \frac{3Elt}{8}$$

and for the diagonal members

$$AE_D = \frac{3Elt\sqrt{2}}{8}$$

where E is the modulus of elasticity of the prototype material, t is the thickness of the section, and l is the side dimension of the unit square frame.

This simple articulated structure is then subjected to the same loading as the beam, and the horizontal and vertical displacements of its various joints are determined by methods applicable to indeterminate structures. The point of the analogy is that these displacements of the joints will be, to a first approximation, the same as the displacements of corresponding points in the solid beam. It is then a matter of simple arithmetic to pass from displacements to strains and thence to stresses. To reach a more accurate determination, the first answer will be improved by dividing the section into a larger number of smaller squares. The advantage of starting with a coarse lattice and then subdividing it will appear later.

The validity of the analogy between the solid section and the corresponding lattice may be demonstrated readily by comparing the behavior of a square element of the solid section with the behavior of a corresponding single lattice square when the two are subjected to the same loads (Fig. 2). It will be found that if the properties of the lattice members are chosen in accordance with the equations given in the preceding paragraph, the displacements of the two will be identical, subject

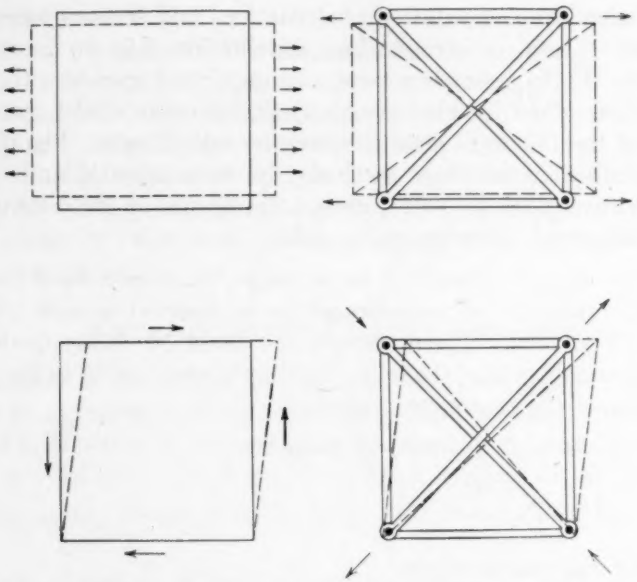


Fig. 2—The lattice frames behave in the same way as solid elements under corresponding loads.

to a restriction that Poisson's ratio for the solid material is $\frac{1}{3}$.* This demonstration of the identity of displacements for equivalent loadings—a demonstration which requires only the application of elementary statics—will be unsatisfactory to those who are accustomed to more rigorous thinking; but for these it may be shown that an iterative method, identical in all respects to the lattice analogy, may be derived by purely mathematical considerations from the fundamental differential equations of the theory of elasticity.

The exact equivalence of the solid section and the lattice applies, of course, only to a few restricted cases—such as the square with a uniform load. For other cases, the section is divided into small elements (squares if possible) and the degree of approximation may be made as close as is desired by increasing the number of such elements.

SIMPLIFIED SOLUTION OF THE LATTICE

The preceding discussion has presented the fundamental principles of the analogy—the representation of a solid section by a simple articulated framework, but the analogy would be of little value without some simple way to solve the lattice for displacements or stresses. A lattice composed of many cross-braced frames is a highly indeterminate structure,

*This restriction on Poisson's ratio is not an essential part of the analogy. The value $\frac{1}{3}$ is used here because the displacement equations turn out to be somewhat simpler for this value than for any other.

and the simplest method of solving it appears to be a relaxation scheme involving the determination of displacements by successive approximations. The problem of the deep beam (Fig. 1) will be solved as an example of the application of the method. The system of computation which seems to be preferable is particularly simple when the boundary conditions of the problem are expressed in terms of displacements rather than loads, and in the present case it is clearly permissible to change from an arbitrary fixed load acting at the loaded joint to an arbitrary fixed vertical displacement—say 100 units—at that joint. The problem then is to determine the horizontal and vertical components of displacement at all of the other joints, and after that is done it is a simple matter to compute the load required to produce the assigned deflection. The two joints at the reactions will be considered fixed against vertical movement, but free to move horizontally.

To visualize the adjustment process, imagine the lattice actually constructed, to a small scale, of elastic members (or springs) with their ends fastened to small rings to form the frictionless joints. The members must, of course, be able to sustain compression without buckling. Lay this lattice out upon a horizontal board, and before applying any displacements fasten each joint to the board by a nail passing through the ring. Then complete constraint of all joints exists, with all displacements zero. Next, as shown by Fig. 3, remove the nail at the

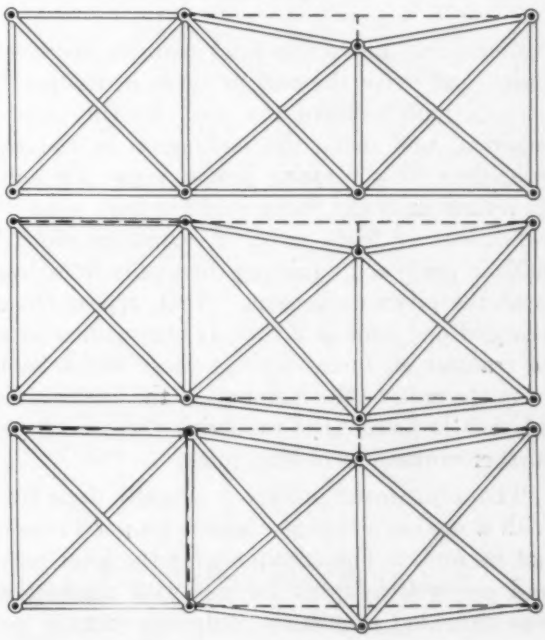
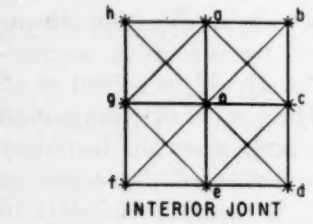
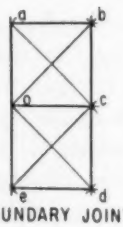


Fig. 3—Steps in the lattice adjustment



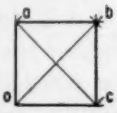
INTERIOR JOINT

$$h_o = \frac{4}{9E} H_o + \frac{1}{12}(h_b + v_b + 4h_c + h_d - v_d + h_f + v_f + 4h_g + h_h - v_h)$$
$$v_o = \frac{4}{9E} V_o + \frac{1}{12}(4v_a + h_b + v_b - h_d + v_d + 4v_e + h_f + v_f - h_h + v_h)$$



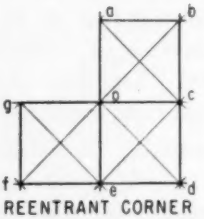
BOUNDARY JOINT

$$h_o = \frac{8}{9E} H_o + \frac{1}{6}(h_b + v_b + 4h_c + h_d - v_d)$$
$$v_o = \frac{8}{9E} V_o + \frac{1}{6}(2v_a + h_b + v_b - h_d + v_d + 2v_e)$$



EXTERIOR CORNER

$$h_o = \frac{2}{3E}(3H_o - V_o) + \frac{1}{4}(-v_a + h_b + v_b + 3h_c)$$
$$v_o = \frac{2}{3E}(H_o + 3V_o) + \frac{1}{4}(3v_a + h_b + v_b - h_c)$$



REENTRANT CORNER

$$h_o = \frac{3}{5E}(H_o - \frac{1}{9}V_o) + \frac{1}{40}(-v_a + 4h_b + 4v_b + 18h_c + 5h_d - 5v_d - 2v_e + 4h_f + 4v_f + 9h_g)$$
$$v_o = \frac{3}{5E}(V_o - \frac{1}{9}H_o) + \frac{1}{40}(9v_a + 4h_b + 4v_b - 2h_c - 5h_d + 5v_d + 18v_e + 4h_f + 4v_f - h_g)$$

NOTES

h, v indicate displacements
H, V indicate forces per unit thickness
E = modulus of elasticity of prototype material

Fig. 4—Equations for the lattice adjustment

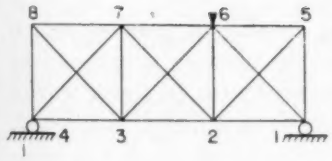
loaded joint, move this joint through the assigned displacement of 100 units, and drive the nail in again in its new location. Then at an adjoining joint remove the nail, let the joint move to its equilibrium position, and drive the nail again in the new location. Repeat the procedure at successive joints, removing and replacing only one nail at a time until the entire grid has been gone over once. Any prescribed conditions of fixity must of course be observed either by leaving the nails in place or by moving them only in certain directions in conformity with the given conditions. Next, repeat the entire process, going over the grid one joint at a time as many times as may be necessary in order to transfer all forces (except those which represent loads or reactions) from the nails to the members of the lattice. Equilibrium will be reached when it is found that removing the nail at any free joint produces no further movement of that joint.

The adjustment process is actually done by computation rather than with a model, although there is no good reason why such models could not be built. The equations for the joint movements are quite simple, and are well adapted for use with modern computing machines. As was indicated previously, only one joint is permitted to move at each

step of the process. Fig. 4 shows the four pairs of equations required for the adjustment of the various types of joints which may be encountered in solving any type of loading: one for a free joint on a straight boundary, one for an exterior corner, one for a reentrant corner, and one for an interior joint. In each case, the horizontal and vertical components of the displacement of the free joint, marked "O", are expressed as a weighted average of the displacements of the surrounding fixed joints plus a constant term whose value depends on the external loads or the body forces acting at the joint.

The positive directions of the coordinate axes in Fig. 4 are taken as shown. If the positive directions are changed, or if the boundaries are located on sides opposite to those shown by the figure, certain signs in the equations will be changed; if the boundaries are at right angles to those shown, the designations h and v will be interchanged, with perhaps sign changes as well. The nature of these changes will be made clear

Fig. 5—Complete computation for the three-frame lattice



8		7		6		5	
h	v	h	v	h	v	h	v
10	2	12	-26	0	-100	-14	-14
17	-1	16	-41	0	-100	-17	-17
22	-4	18	-50	0	-100	-18	-18
25	-5	20	-56	0	-100	-18	-18
28	-6	22	-60	0	-100	-17	-17
30	-7	23	-62	0	-100	-17	-17
31	-7	24	-64	0	-100	-16	-16
33	-8	25	-65	0	-100	-16	-16
34	-8	26	-66	0	-100	-16	-16
34	-8	26	-67	0	-100	-16	-16
35	-8	27	-67	0	-100	-16	-16
35	-8	27	-67	0	-100	-16	-16
400	-118	314	-695	0	-1000	-160	-234※

4		3		2		1	
h	v	h	v	h	v	h	v
-9	0	-13	-17	11	-67	33	0
-15	0	-15	-35	11	-78	41	0
-18	0	-15	-47	12	-82	41	0
-20	0	-14	-54	14	-84	41	0
-21	0	-13	-59	16	-85	43	0
-21	0	-12	-62	18	-86	44	0
-20	0	-11	-64	20	-86	45	0
-20	0	-10	-66	21	-87	47	0
-19	0	-9	-67	22	-87	47	0
-19	0	-8	-68	23	-87	48	0
-19	0	-8	-68	24	-88	49	0
-19	0	-8	-68	24	-88	49	0
-209	0	-68	-698	259	-843	546	0※

※ These figures from 48-square lattice

by a little study of the diagrams and the corresponding equations. Occasionally, other simple alterations may have to be made in the fundamental equations. For example, consider the joint at the left reaction of the beam being analyzed. Due to the assigned conditions at this joint the value of v_o is known ($v_o = 0$) while that of V_o is unknown. Elimination of V_o, using also the given condition that $H_o = 0$, yields

$$h_o = \tfrac{1}{3}\,(h_b + v_b + 2h_c)$$

$$v_o = 0$$

Fig. 5 shows the complete computation for the displacements of the deep beam simulated by a coarse lattice. The units represented by the figures are, of course, entirely arbitrary—they may be millionths or thousandths of an inch, or any other value depending on the choice of dimensions and elastic properties of the beam. The time required for this computation was about 1½ hours. It will be noted that in progressing down the columns—that is, in following the successive steps in the computation of displacements—the differences between successive computations become smaller and smaller until finally the equilibrium condition is reached, at which no further change occurs. The final values in each column, below the horizontal lines, were arrived at by subdividing the three-square lattice into a 48-square lattice and again carrying the computation to equilibrium. It will be noted that if the three-square values are taken as a starting point for the finer grid, then only minor adjustments are required to reach the more accurate values. In this illustrative example the computation was carried out as a completely automatic procedure, with the computer doing nothing but performing simple arithmetic operations on two- or three-figure numbers. However, a great amount of time may be saved if the computer will use his ingenuity by introducing intelligent guesses which speed up the convergence. This is permissible because a convenient characteristic of the process is that the final answer must be right, within some specified limit of error, regardless of the path by which it is reached. This characteristic also means that there is no need to check the computations, for an error made in any step will automatically be eliminated by the later steps.

It is obvious that the differences between the final displacements of adjacent joints give the strains. With the strains known, the conventional stress-strain relationships for elastic materials are applied to obtain the stresses. The stresses shown in Fig. 6 were computed from the 48-square lattice. This figure also shows the horizontal stresses and the horizontal-vertical shear stresses computed by the elementary methods. The weight of the beam was omitted to keep the illustrative example as simple as possible. This example was not chosen as a spec-

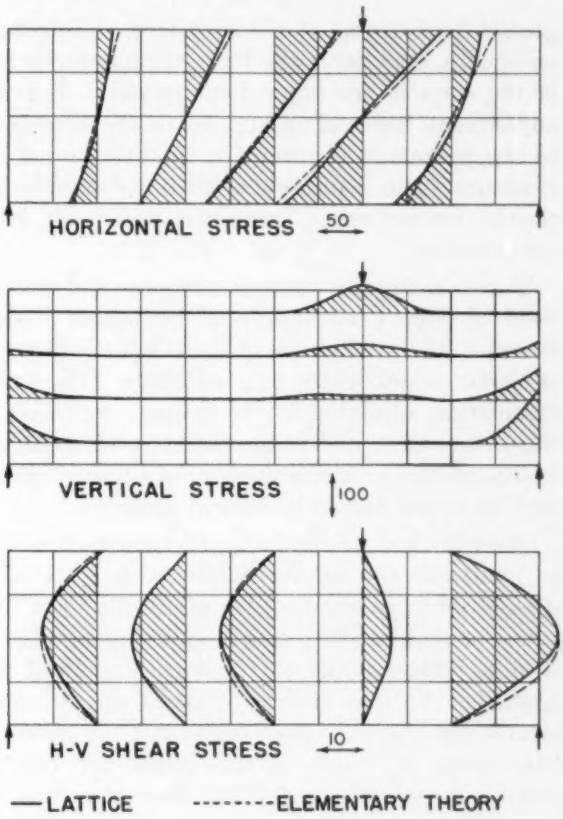


Fig. 6—Stresses in the deep beam

tacular one, but rather as a common one which, from the standpoint of precise mathematics, presents considerable difficulty. In this simple case the differences between stresses computed by elementary theory and those computed from the lattice are not at all startling. Some points of interest could be brought out, particularly in connection with the deflections and the principal stresses and maximum shears, but the present discussion is concerned with the method of computation, not with the results.

VARIATIONS IN APPLICATION

Most concrete structures rest on elastic foundations, and frequently the foundation displacements have an important bearing on the stresses in the structure. If the foundation is assumed to be rigid, it is taken into account simply by assigning zero displacement to all points at the base of the structure. In the case of a yielding foundation, equations are available from the theory of elasticity for computing the displacements of the boundary of an infinite half-plane due to normal or tan-

gential loads acting at the boundary. With the lattice structure resting on such a boundary, the base displacements may be computed as one of the steps in the adjustment process. It is evident that on the first adjustment the loading applied to the boundary will be incorrect, but as the process is continued a condition must be reached in which the structure is in equilibrium with its foundation. Differences in the elastic properties of the two will not introduce any additional complication.

If two materials such as concrete and metal are joined, the equations of Fig. 4 will apply within either material. At the junction, however, the coefficients of these equations will depend upon the ratio of elastic moduli of the two materials. They may be derived readily by considering adjoining lattice frames, with the members of each frame chosen to correspond to the material which they represent. It may also be feasible to let a single member of a frame represent a different material, such as a steel bar embedded in concrete.

Other devices will be fairly obvious, such as the introduction of cracks or joints, or the representation of a joint which will transmit compression but not shear. Still others may tax the ingenuity of the computer, or require the use of simplifying assumptions or the introduction of new combinations of the lattice method with other more formal methods. Various devices to speed up the rate of convergence of the process can also be worked out to suit the taste of each computer. Group relaxations, in which several joints are treated simultaneously, will greatly increase this rate, but they require rather involved equations. In a discussion of the method, G. M. J. Williams* has proposed that horizontal and vertical displacements be treated simultaneously rather than separately, and an extension of the method to handle three-dimensional cases involving axial symmetry has recently been developed by Guthlac Wilson.†

*Williams, G. M. J., Discussion of "The Stress-Distribution in Gravity Dams," *Journal* Institute of Civil Engineers, Supplement, Oct. 1947. pp. 481-483.

†Wilson, Guthlac, "A Relaxation Method for the Solution of Problems Concerning Axially Symmetrical Distributions of Load in an Elastic Medium", *Journal* Institute of Civil Engineers, April 1948, pp. 149-166.

Strength of Precast Concrete Floor Joists*

By JACOB FELD†

SYNOPSIS

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

INTRODUCTION

The design for the first floors of a very large housing project of two-and three-story multiple dwelling units consisted of 8-in. precast concrete joists spaced 2 ft 9 in. apart, with a clear span of 11 ft 7 in., one end resting on a concrete block wall with 4 in. bearing, the other end embedded 4 in. in a longitudinal poured concrete girder 10 in. wide. The buildings were of varying lengths, and two spans in width. The floor slab (Fig. 1) was 2½ in. poured concrete into which the tops of the joists were embedded ½ in. The slab and the girders were poured monolithically.

After a small part of the contract was performed, including the pouring of some floor slabs, setting forms and joists for other floors, and stockpiling a great number of joists in irregular piles (Fig. 2) (as if dumped from flat trucks), the project was abandoned. After about a year, a new contract was awarded to complete the project. One of the problems assigned to the writer was to determine whether the stockpiled joists were usable and how many could be salvaged.

TYPE OF JOISTS

Precast concrete joists were Lith-I-Bar type, made with lightweight aggregate, 8 in. deep and 12 ft long. Reinforcement consisted of

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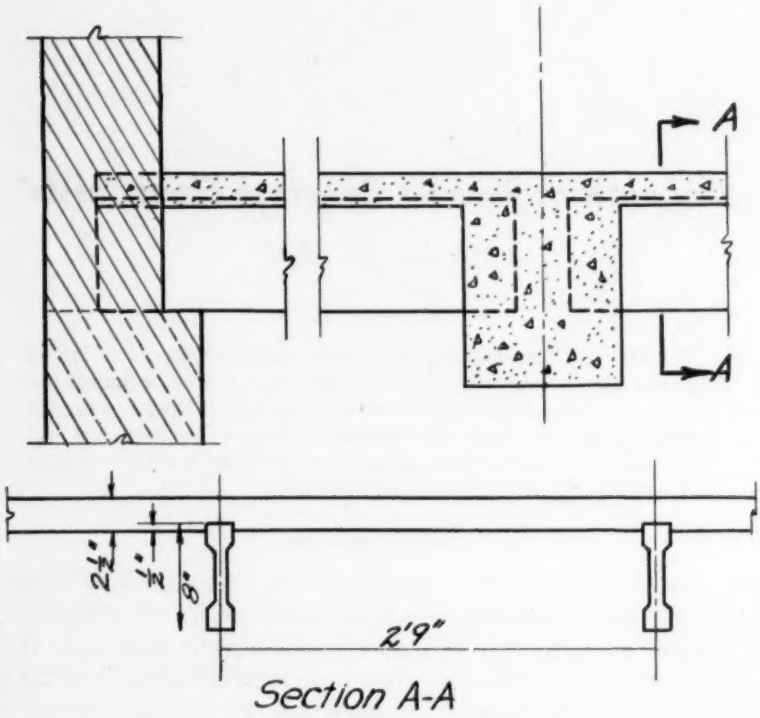

Fig. 1—Typical floor construction using the precast joists

two ½-in. round bars in the bottom and one ⅜-in. round top bar with diagonal shear bars welded to both top and bottom bars. Tests had been made of a floor slab section using four new ribs in the casting yard, with the same span and spacing as designed for the buildings. The results were as follows (11.58 ft clear span, 2.75 ft spacing):

Live load, lb per sq ft	Total load, lb per sq ft	Deflection, ft
30	71	0.0011
60	101	0.0026
90	131	0.0052
120	161	0.0076
150	191	0.0105
180	221	0.0145

At this loading, no cracks were visible in the joists. Fine tensile cracks appeared in the bottom of the joists near mid-span at a live load of 240 lb per sq ft, with a corresponding deflection of 0.036 ft. The design live load was 40 lb per sq ft.

Fig. 2—Precast joists as stockpiled

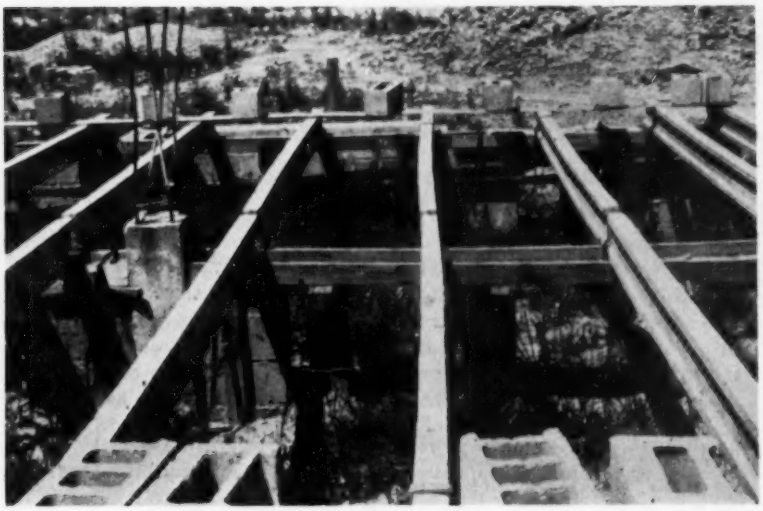

Fig. 3—Typical joist set-up in floor

CONDITION OF THE JOISTS

A joint inspection by all men concerned was arranged. In the buildings ready for the first floor a considerable number of joists were found to contain cracks, usually five or six in a distance of about 6 ft in one half the length of the joist. A great number of these joists were also bowed or S-shaped, the amount of deviation from

a straight line varying from ½ in. to a maximum of 1½ in. In some of the buildings the ends of joists resting on the wood form for the center girder had the lower corners chipped off, reducing the amount of bearing in the future concrete girder to almost zero for the lower flange. However, the upper flanges were whole. (See Fig. 3.)

The joists stocked for future use, without exception, were bowed 1 in. to 1½ in. from a straight line, and always in the same direction. These joists are manufactured in a flat position; the underside in contact with the form is not as smooth as the top face which has been troweled. In all cases the bow is equivalent to a curling upwards of the ends of the joists, as they were positioned in the mold.

It was agreed at the inspection that there was no definite or regular consistency in the locations of hair line cracks, that the cause was chiefly a volume change, not extra loading or the handling of the materials.

It was also agreed that these joists are welded steel joists and the concrete of the bottom chord is placed only as a fireproofing. Therefore there is no objection to the use or reuse of the joists which have hair line cracks or of the joists with chipped ends. A full embedment of the top chord of a joist in the concrete girder is sufficient to transfer any expected load from the joist to the girder.

CONDITIONS OF ACCEPTANCE

The managing architect then agreed to permit the use of the joists on the following conditions:

1. A load test to be made on five joists picked indiscriminately from the materials on the site, spaced 33 in. on centers with 2½ in. concrete slab constructed thereon, the bottom of the slab to be ½ in. below the top of the joist. The test load is to be 100 lb per sq ft uniformly distributed and may be made up of sand, cement, blocks or any other material. If high early strength cement is used, the test can be performed 7 days after the slab is constructed. It is understood that the slab is to be wet cured for 2 days after it has set.
2. The maximum deflection of the joists under the test load was not to exceed $\frac{0.001\ L^2}{t}$ ft, where L is the span in feet and t is the depth in inches. For the conditions of the test, this meant a maximum deflection of 0.016 ft.

To avoid any possible argument, each of the six interested control agencies and departments was requested to pick one joist. Needless to say, the best ones were not represented in the test panel.

It was further agreed, that if the load test requirement was met, the definition of usability would be limited to the following:

1. Joists which had been placed upside down must be removed and turned right side up.
2. Hair line cracks in joist are not to be a criterion for rejection.

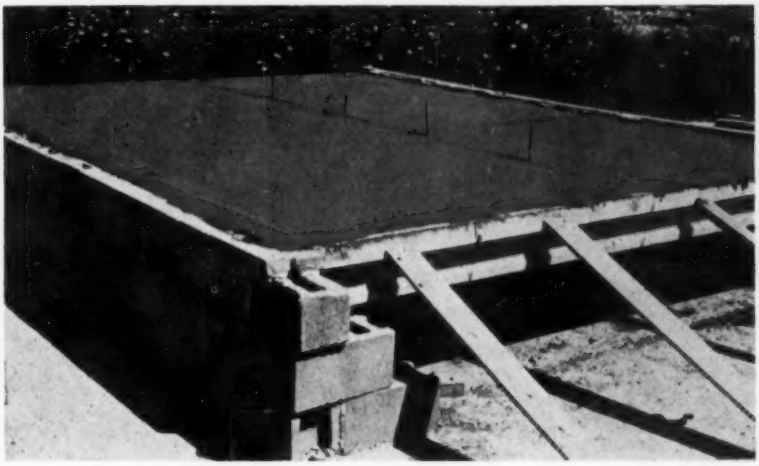

Fig. 4—Test panel. Note the rods over the joists for measuring deflections.

3. Excessive warping or bowing is a criterion for rejection. The maximum tolerance for deviation from a straight line for the web of the joist is ½ in.
4. For joists in place and bowed more than ½ in., reuse will be permitted if spacers or ties are provided to bring the joist within the ½-in. tolerance before pouring the 2½-in. slab.
5. The joists stocked for future use and bowed beyond the ½ in. tolerance may be used in the buildings under the same conditions as above.

LOAD TEST

The chosen joists were set up as a section of floor and the slab poured to simulate the required conditions (see Fig. 4). The results of the load test under 100 lb per sq ft superimposed load and a dead load of 41 lb per sq ft are given in the accompanying table.

Joist	Condition of joist before testing	Deflections (ft) after 24 hr, loaded	Deflections (ft) after unloading
A	7 Hair cracks	0.003	0.002
B	5 Hair cracks	0.003	0.000
C	6 Hair cracks	0.008	0.004
D	1 Hair crack and broken top flange	0.011	0.005
E	5 Hair cracks	0.002	0.000
F	10 Hair cracks	0.006	0.003

CONCLUSIONS

Since the load test requirements were fully met, all of the joists were used and a considerable saving resulted. An inspection of the buildings after two years use showed no structural defects of any kind.

Three to four percent of entrained air in concrete does not always provide the high resistance to freezing and thawing described in recent years. An exception is here discussed with the use of "sand-gravel" aggregates.

Effect of Entrained Air on Concretes Made with So-Called "Sand-Gravel" Aggregates*

By PAUL KLIEGER†

SYNOPSIS

Sand-gravel aggregates (maximum particle size ⅜ in.) used with *non-air-entraining* cement produce concretes containing from 3 to 4 percent air. The same mixes made with *air-entraining* cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

INTRODUCTION

In certain areas of Kansas, Nebraska, northwestern Missouri and western Iowa, where natural coarse aggregates are scarce, so-called "sand-gravel" aggregates are commonly used in the production of concrete for construction projects. These aggregates have a maximum particle size of ⅜ in. and contain from 80 to 95 percent of 0 to No. 4 material by weight.

The use of these sand-gravel aggregates constitutes a problem of particular interest in the field of air-entraining concrete. When used with non-air-entraining portland cement in mixes containing 6½ sacks per cu yd and having a slump of 3 ± ½ in., they produce concretes containing from 3 to 4 percent of air. Although this amount of air provides a high degree of resistance to freezing and thawing for the usual concrete containing aggregates graded up to 1½-in. maximum size, it does not give the desired resistance in concretes with sand-gravel aggregates. However, when used with air-entraining portland cement in mixes of 3-in. slump containing 6½ sacks per cu yd, these aggregates

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TABLE 1—CHEMICAL COMPOSITION, POTENTIAL COMPOUND COMPOSITION AND MISCELLANEOUS TESTS OF CEMENTS

Chemical Composition

Cement	Major components—percent						
	SiO_2	Al_2O_3	Fe_2O_3	Total CaO	MgO	SO_3	Ignition loss
17775-6	21.05	5.11	2.99	63.94	3.11	2.23	0.91

Cement	Minor components—percent						
	Mn_2O_3	Free CaO	Ins. res.	Alkalies			
				Na_2O	K_2O	Total	Total as Na_2O
17775-6	0.14	0.34	0.17	0.23	0.39	0.62	0.49

Potential Compound Composition

Free CaO deducted from total CaO

Cement	Type	Calculated compound composition—percent						Chloroform soluble material, percent
		C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO	
17775	I	54	20	8	9	3.8	0.34	Trace
17776	IA	54	20	8	9	3.8	0.34	0.021

Miscellaneous Tests

Cement	Type	Specific gravity		Passing No. 325 Sieve, percent	Specific surface (Wagner) cm^2 per g
		Kerosene	Water		
17775	I	3.157	3.217	86.6	1720
17776	IA	3.153	3.218	87.3	1760

produce concretes having air contents ranging from 8 to 13 percent, depending on the grading of the aggregate. The effect of this increased amount of entrained air on the resistance to freezing and thawing, water-cement ratio, plasticity, bleeding, strength and length change of concretes made with typical "sand-gravel" aggregates from four different sources is discussed in this report.

MATERIALS

Cement

The Type I and Type IA portland cements for this study were obtained from a Kansas plant. Both cements were ground from the same Type I clinker. Vinsol resin in the form of sodium resinate was used in grinding the Type IA cement. Table 1 shows the chemical

composition, potential compound composition and results of miscellaneous tests of the cements.

The air contents of 1:4 standard sand mortars made with four combinations of the Type I and Type IA cements are shown in the last column of Table 2. These tests were made in accordance with A.S.T.M. C185-47T.

The amount of air entrained (19.9 percent) in the standard sand mortar with Type IA cement meets the requirement of 18 ± 3 percent of A.S.T.M. C175-47T. The amount of air entrained with Type I cement was 4 percent. Replacing one-third of the Type I cement with Type IA cement tripled the air content of the standard sand mortar.

Table 2 shows 7 and 28-day tensile and compressive strengths of mortars made in accordance with A.S.T.M. standard methods C190-44 and C109-44 respectively. The tensile and compressive strengths of the mortars made with the Type IA cement at 7 days were 90 percent and at 28 days 85 percent of the corresponding strengths of the mortars made with the Type I cement. The blends of the two types of cement gave intermediate strength values.

Aggregates

The sand-gravel aggregates were obtained from the following sources and were used in the grading as received:

1. Platte River, South Bend, Nebraska, Lot 17701.
2. Platte River, Schuyler, Nebraska, Lot 17702.
3. Republican River, McCook, Nebraska, Lot 17703.
4. Florena Switch, Kansas, Lot 17834.

The Platte River sand-gravel from South Bend, Nebraska, was also used in combination with 30 percent by weight of a soft limestone coarse aggregate (graded No. 4 to 1 in.) from the Chicago, Illinois area. The

TABLE 2—STRENGTH TESTS OF CEMENTS

Cements blends made with Type I and Type IA portland cements.
Tests for compressive strength of 2-in. plastic mortar cubes were made in accordance with A.S.T.M. C109-44; those for tensile strength of briquettes were made in accordance with A.S.T.M. standard method C190-44.
Air content of mortar determined in accordance with A.S.T.M. C185-47T.
All strength specimens were cured 24 hours in molds in moist closet, then in water until tested at age indicated. Each value is the average of three tests.

Amount of Type IA cement in blend, percent	Tensile strength, 1:3 standard Ottawa sand mortar briquettes, psi				Compressive strength of 1:2.75 graded Ottawa sand mortar cubes (2-in.), psi				Air content of 1:4 standard sand mortar, percent
	7-day	percent	28-day	percent	7-day	percent	28-day	percent	
0	435	100	560	100	2880	100	4520	100	4.0
33⅓	430	99	525	94	2690	93	4150	92	12.7
66⅔	420	97	475	85	2560	89	3670	81	17.4
100	390	90	475	85	2580	90	3800	84	19.9

gradation of the aggregates and results of other physical tests are shown in Table 3.

Handling and mixing

Each sand-gravel aggregate was used with each of four cement combinations in mixes containing 6½ sacks of cement per cu yd of concrete and having a slump of 3 ± ½ in. These cement combinations were as follows:

Combination	Cement, percent	
	Type I	Type IA
1	100	0
2	66⅔	33⅓
3	33⅓	66⅔
4	0	100

In addition, the Platte River sand-gravel aggregate from South Bend, Nebraska, was combined with 30 percent of No. 4 to 1-in. Chicago limestone by weight and used in a concrete mix having the same water-cement ratio and slump as that used with the sand-gravel alone.

The aggregates were air-dried, separated into six or more sizes, and recombined when the batches were weighed to give the same grading as when received. The air-dried aggregates were weighed and then soaked under water for 24 hours prior to mixing in the concrete.

The concrete was mixed for 2½ minutes in an open-tub type Lancirick mixer of 1½ cu ft capacity. Each batch produced 1 cu ft of concrete.

TESTS MADE ON FRESH CONCRETE

Tests of fresh concrete included slump, yield and direct determinations of air content by the pressure method[1]. Tests on the hardened concrete included air void observations by the camera lucida method[2], flexural and compressive strengths at 7 and 28 days, resistance to freezing and thawing, and length change.

Water-cement ratio, plasticity and bleeding determination

Table 4 shows the cement content, net water-cement ratio and slump of the different concretes.

The use of Type IA cement made possible reductions in net water-cement ratios ranging from ½ to 1¾ gallons per sack of cement. For each aggregate the reduction in net W/C increased with increase in air content of the concrete.

The concretes made with Type IA cement were considerably more plastic and workable than those made with the Type I cement. Tendencies toward segregation in some of the Type I concretes were entirely eliminated in the concretes made with Type IA cement.

TABLE 3—PHYSICAL CHARACTERISTICS OF SAND-GRAVEL AGGREGATES

Lot No.	Source	Percent retained—cumulative									Fineness modulus	Spec. grav., bulk ssd*	24-hour absorp. percent by wt
		1 in.	¾ in.	⅜ in.	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100			
17834	Florena Switch, Kansas	0	0	3	20	47	71	87	93	98	4.19	2.58	1.36
17701	Platte River, South Bend, Nebr.	0	0	2	16	39	76	92	97	99	4.21	2.60	0.91
17702	Platte River, Schuyler, Nebr.	0	0	5	21	49	70	80	90	98	4.13	2.60	0.90
17703	Republican River, McCook, Nebr.	0	0	0	7	31	64	84	96	99	3.81	2.57	1.37
17838	Chicago Limestone	0	28	88	100	100	100	100	100	100	7.16	2.61	2.13
17701 +30 percent by wt. 17838	S. Bend, Nebr.+ Chicago Limestone	0	6	19	33	51	81	94	98	99	4.81	——	——

*Saturated surface dry

Bleeding of the concretes was reduced considerably by the use of Type IA cement. The bleeding of the concretes made with the Type I cement ranged from slight to considerable (⅛ in. of water on surface of a 6 x 6 x 30-in. beam).

Air content of fresh concretes

Table 4 shows air content of the concretes as determined by both the pressure and the gravimetric method[3]. In general the air contents determined by the pressure method were slightly higher than those determined by the gravimetric method.

Fig. 1 shows the air content of the concretes made with the different aggregate and cement combinations. Concretes made with Type I cement had approximately the same air content (3 to 4 percent) for the four sand-gravel aggregates, while with Type IA cement the air contents ranged from 8 to 12 percent. Concretes made with the blends of Type I and Type IA had intermediate air contents.

TABLE 4—CHARACTERISTICS OF FRESH CONCRETE MADE WITH SAND-GRAVEL AGGREGATES AND TYPES I AND IA PORTLAND CEMENTS

Cement blends made with Type I and Type IA cements.
Nominal slump: 2 to 3 in.
Unit weight and air content of concrete by gravimetric method determined with a 4 x 8 x 12-in. yield cone.

Ref. No.	Amount of Type IA cement in blend, percent	Tests of fresh concrete					
		Cement content sacks per cu yd	Net W/C, gal./sack	Average slump, in.	Average air content, percent		Unit weight, lb per cu ft
					Pressure	Gravimetric	
		Aggregate 17834					
1	0	6.5	5.6	3.3	3.0	3.0	142.7
3	33⅓	6.5	5.2	3.5	6.3	6.2	138.8
4	66⅔	6.5	4.8	2.9	8.4	8.0	137.0
2	100	6.6	4.6	2.5	9.7	9.3	135.5
		Aggregate 17701					
5	0	6.5	6.3	2.7	3.3	3.4	140.6
7	33⅓	6.5	5.2	2.5	9.1	8.6	135.7
8	66⅔	6.5	4.7	2.5	11.0	10.7	133.8
6	100	6.4	4.6	2.6	12.4	12.4	131.7
		Aggregate 17702					
9	0	6.5	5.2	2.5	3.1	3.3	144.8
11	33⅓	6.5	4.9	2.2	5.3	5.2	142.7
12	66⅔	6.5	4.7	2.3	6.8	6.8	140.5
10	100	6.5	4.7	2.4	8.3	8.1	138.8
		Aggregate 17703					
13	0	6.5	6.0	2.0	4.2	3.8	140.2
15	33⅓	6.4	5.6	2.7	8.6	7.9	135.0
16	66⅔	6.4	5.1	2.6	11.0	10.3	132.4
14	100	6.4	4.9	2.1	11.9	11.3	131.6
		Aggregate 17701 plus 30 percent by weight aggregate 17838					
17	0	5.8	6.1	2.7	1.5	2.2	144.9
19	33⅓	6.3	5.1	3.2	5.5	5.7	141.4
20	66⅔	6.6	4.6	3.6	8.2	8.0	138.7
18	100	6.2	4.4	2.5	9.2	9.2	137.9

The addition of limestone coarse aggregate as a "sweetener" to the South Bend sand-gravel aggregate resulted in a coarser gradation and reduced the air content of the mix made with Type I cement from 3.3 percent to 1.5 percent, and of the mix made with Type IA cement from 12.4 percent to 9.2 percent.

Although the fineness modulus of these sand-gravel aggregates was nearly the same there was a considerable range in the air content of the concretes. For these particular concretes there appears to be a

Fig. 1 (right)—Air contents of concrete made with blends of Types I and IA portland cements and sand-gravel aggregates

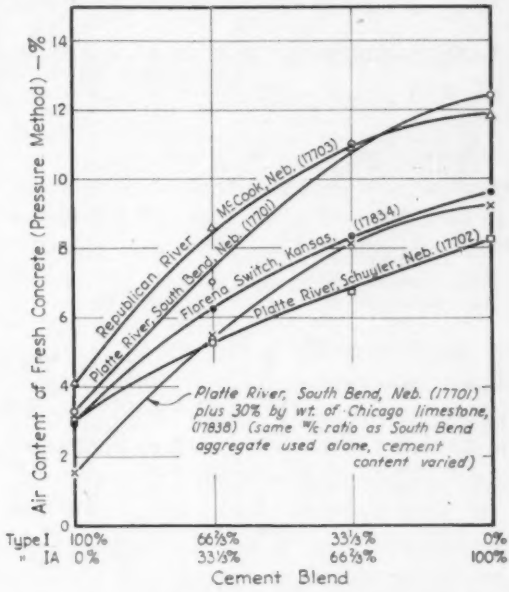

Air Content of Fresh Concrete (Pressure Method) — %
0
2
4
6
8
10
12
14
Republican River
Mc Cook, Neb. (17703)
Platte River, South Bend, Neb. (17701)
Florena Switch, Kansas (17834)
Platte River, Schuyler, Neb. (17702)
Platte River, South Bend, Neb. (17701) plus 30% by wt. of Chicago limestone, (17838) (same w/c ratio as South Bend aggregate used alone, cement content varied)
Type I 100% 66⅔% 33⅓% 0%
" IA 0% 33⅓% 66⅔% 100%
Cement Blend

Fig. 2 (below)—Observed air voids in concretes made with sand-gravel aggregate from South Bend, Nebraska

Cement content—6½ sacks per cu yd, nominal slump, 2 to 3 in. Indicated air contents were determined by the pressure method.

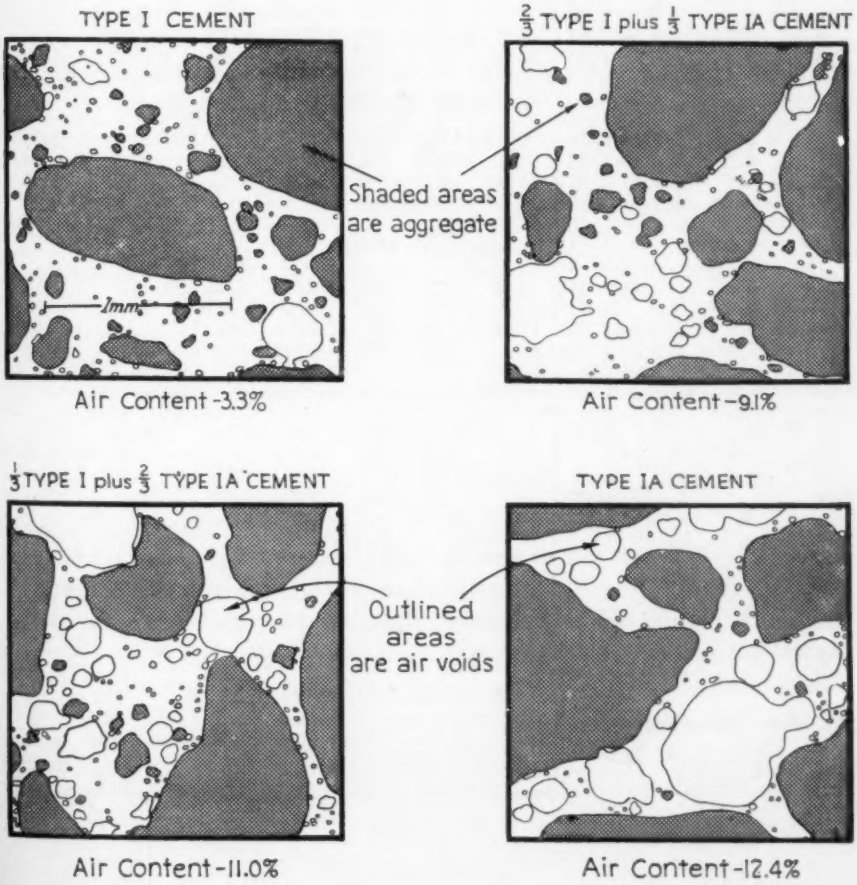

TYPE I CEMENT
⅔ TYPE I plus ⅓ TYPE IA CEMENT
1mm
Shaded areas are aggregate
Air Content -3.3%
Air Content -9.1%
⅓ TYPE I plus ⅔ TYPE IA CEMENT
TYPE IA CEMENT
Outlined areas are air voids
Air Content -11.0%
Air Content -12.4%

TABLE 5—STRENGTH TESTS OF 6 x 6 x 30-IN. CONCRETE BEAMS MADE WITH SAND-GRAVEL AGGREGATES

Flexure: Center load, 18-in. span. Two breaks per beam.
Compression: Modified cubes.
Air-cured specimens soaked in water 48 hours prior to test.
Nominal slump—3 in.
All results are the averages of six individual tests.

Ref. No.	Cement content, sacks per cu yd	Net W/C gal. per sack	Void-cement ratio	Air cont., percent, (pressure)	Specimens cured in moist room				Specimens cured 7 days moist, then in air of lab	
					Modulus of rupture, psi		Compressive strength, psi		28 days	
					7-day	28-day	7-day	28-day	Mod. rupt.	Comp.
Aggregate 17834										
1	6.5	5.6	1.85	3.0	550	665	3520	5110	600	4800
3	6.5	5.2	2.05	6.3	515	640	3580	4780	610	4700
4	6.5	4.8	2.10	8.4	520	645	3350	4640	645	4480
2	6.6	4.6	2.15	9.7	505	595	3230	4280	650	4280
Aggregate 17701										
5	6.5	6.3	2.10	3.3	545	670	2740	4170	515	3770
7	6.5	5.2	2.26	9.1	575	670	3340	4400	585	3920
8	6.5	4.7	2.30	11.0	595	655	3180	4170	580	4030
6	6.4	4.6	2.42	12.4	575	670	3060	3910	590	3780
Aggregate 17702										
9	6.5	5.2	1.77	3.1	620	725	4180	5720	645	5930
11	6.5	4.9	1.85	5.3	610	715	4150	5360	690	5540
12	6.5	4.7	1.95	6.8	625	715	4190	5150	635	5080
10	6.5	4.7	2.05	8.3	590	690	3570	4910	640	4670
Aggregate 17703										
13	6.5	6.0	2.04	4.2	575	650	3270	4520	585	4650
15	6.4	5.6	2.30	8.6	535	670	3160	4130	625	4220
16	6.4	5.1	2.40	11.0	510	605	3110	3760	550	3740
14	6.4	4.9	2.41	11.9	545	600	2990	3900	590	3770
Aggregate 17701 plus 30 percent by weight of aggregate 17838										
17	5.8	6.1	1.97	1.5	590	725	3180	4900	665	4760
19	6.3	5.1	1.97	5.5	665	775	3910	5660	760	5590
20	6.6	4.6	2.01	8.2	675	760	4090	5360	725	4930
18	6.2	4.4	2.11	9.2	690	735	3710	4940	740	4720

relationship between the air content and the amount of material between the No. 48 and No. 100 sieves. Even though the range is small the aggregates having the least amount of material in this size produced the highest air contents in the concretes.

TESTS OF THE HARDENED CONCRETE

Air voids observed by camera lucida

Air voids were observed by the camera lucida method in hardened concretes made with Platte River sand-gravel from South Bend,

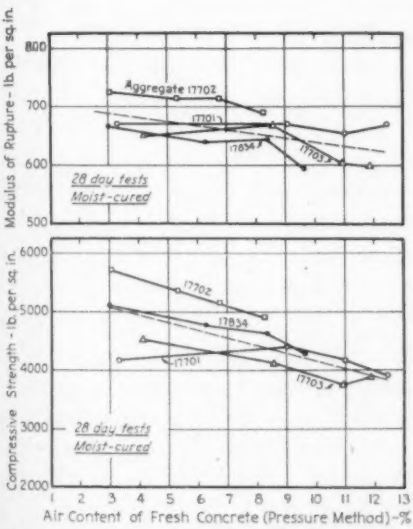
Nebraska. Fig. 2 shows a typical observation field for each of the four concretes made with this aggregate and with the Type I and Type IA cements and the blends of these two cements.

The air voids in the concrete made with the Type I cement appear to be of two sizes. There are a few large voids and a considerable number of very small air voids, while in the concretes containing Type IA cement there were in addition a large number of air voids of intermediate sizes distributed throughout the paste.

Strength test results

Flexure and compression—Strengths of 6 x 6 x 30-in. concrete beams at ages of 7 and 28 days are presented in Table 5. The beams were cured 1 day in molds and then as indicated in the table. They were tested in flexure with the load applied at the center of an 18-in. span. Two tests were obtained on each beam. Compression tests were made on the beam ends by the modified cube method.

The effect of air content on the 28-day flexural and compressive strength of concretes made with the sand-gravel aggregates is shown in Fig. 3 and 4. Similar relationships were obtained at 7 days. Fig. 3 shows results of 28-day tests of concretes given continuous moist curing. Fig. 4 shows results of 28-day tests of concretes cured 7 days moist, 19 days in the air of the laboratory, and 2 days in water. In general the relationships indicate a decrease in strength with increase in the amount



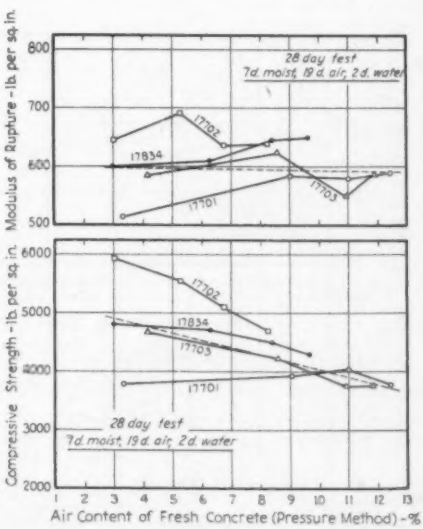


Fig. 3 (left)—Effect of air content on strength of moist-cured concrete made with sand-gravel aggregates and blends of Types I and IA portland cements

Fig. 4 (right)—Effect of air content on strength of air-cured concrete made with sand-gravel aggregates and blends of Types I and IA portland cements

of entrained air. The decrease in compressive strength averages about 2½ percent for each percent increase in air content of both the moist-cured and air-cured concretes. Average flexural strength decrease is slightly over 1 percent for each percent increase in air content for moist-cured concrete. For air-cured concrete there was little, if any, loss in flexural strength with increased air content.

Relationships between the voids-cement ratio[4] and strength are shown in Fig. 5. In general, an increase in the voids-cement ratio* was accompanied by a decrease in both flexural and compressive strength. The decrease was more pronounced for compressive strength than for flexural strength. For these concretes of constant cement content, the voids-cement ratio increased with an increase in air content, despite sizeable reductions in water content made possible by the increased air content.

With few exceptions, the strengths of concretes moist-cured 28 days are higher than the strengths of concretes cured 7 days moist, 19 days in the air of the laboratory and 2 days in water, as shown in Table 5.

Effect of coarse aggregate addition—In those regions where sand-gravels occur in abundance and where coarse aggregates are scarce, it is sometimes the practice to add small amounts of coarse aggregate as a so-called "sweetener". Results for mixes made in this manner are

*The voids-cement ratio is the ratio between the absolute volume of the total voids in the concrete (air + water) and the absolute volume of the cement.

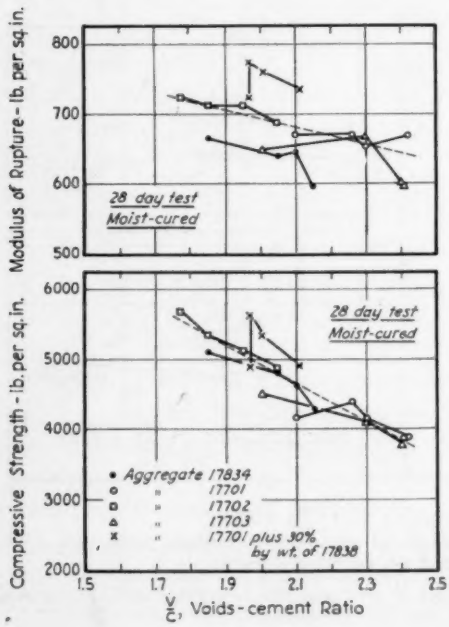


Fig. 5—Relationships between voids-cement ratio and strength of concretes made with sand-gravel aggregates and blends of Types I and IA portland cements

shown in Table 5. These data show the strengths of concretes made with the South Bend, Nebraska sand-gravel to which 30 percent of limestone coarse aggregate, by total weight of aggregate, had been added.

These concretes had slightly lower net water-cement ratios than the concretes made with the sand-gravel aggregate alone. The addition of the limestone, resulting in a coarser gradation, reduced the air contents of the concretes and thereby reduced the voids-cement ratio. The reduction in voids-cement ratio is reflected in the strengths. In all cases, the concretes with the added limestone coarse aggregate showed higher strengths in flexure and compression than the corresponding concretes without the added limestone.

Freezing and thawing

Concrete prisms, 3 x 3 x 11¼ in., equipped with stainless steel gage plugs set in the ends, were made from the same batches as the 6 x 6 x 30-in. beams. These prisms were cured 1 day in molds, 13 days in the moist room, 14 days in the air of the laboratory at 75 F and 50 percent relative humidity, and 3 days in water prior to freezing and thawing. Freezing and thawing tests (2 complete cycles every 24 hrs) were conducted with the prisms immersed in tap water. The resistance to freezing and thawing was evaluated by measurements of linear expansion, change in dynamic E and change in weight, which are plotted in Fig. 6.

The diagrams in this figure illustrate in a striking manner the great increase in resistance to freezing and thawing resulting from the use of Type IA cement with these sand-gravel aggregates. All of the concretes made with Type I cement show considerable expansion and drop in dynamic E during 50 cycles of freezing and thawing. At 100 cycles, these concretes either show expansions above 0.20 percent and reductions in dynamic E greater than 50 percent, or have disintegrated. The concretes made with the blends of Types I and IA or with Type IA alone all show low expansion and weight loss and very little change in dynamic E during 200 cycles of freezing and thawing.

Concretes made with the Republican River aggregate from McCook, Nebraska, and air-entraining cement show slightly greater expansions and definitely greater reductions in dynamic E than concretes made with the other three sand-gravel aggregates.

Fig. 7 shows the relationship between the air content of fresh concrete and the percent expansion during 100 cycles of freezing and thawing. This relationship indicates that air contents above 6 percent (about 3 percent higher than the air contents normally obtained with these sand-gravels and non-air-entraining cements) will insure greatly in-

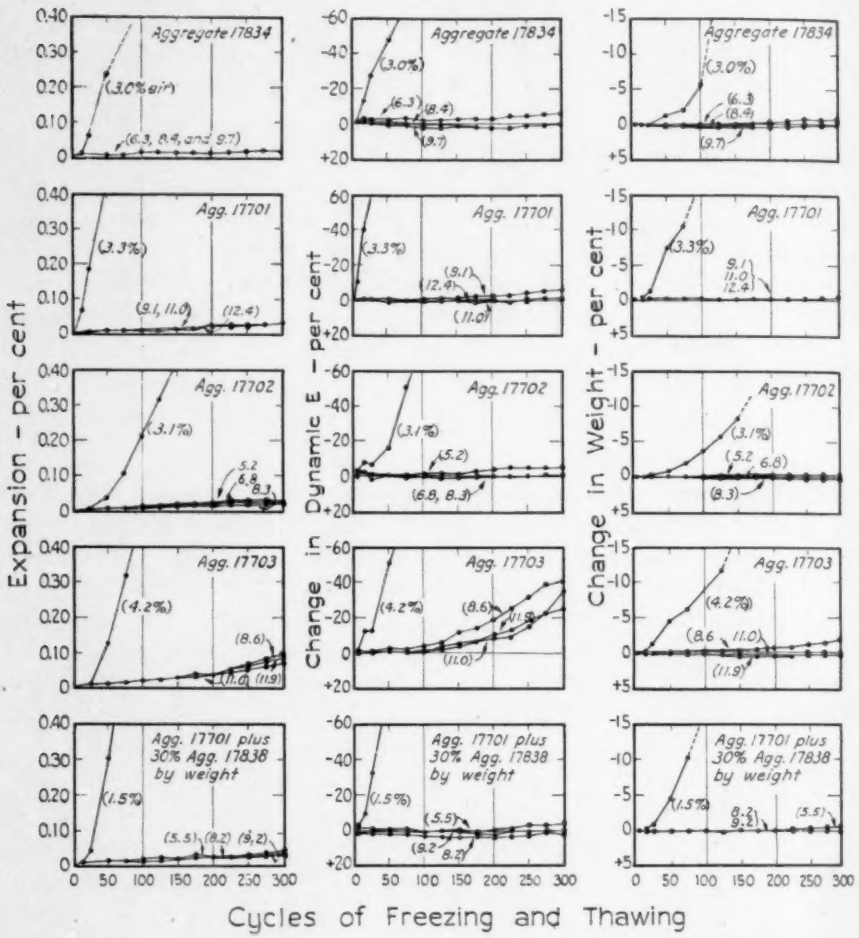


Fig. 6—Percent expansion, drop in dynamic E and loss in weight of sand-gravel concretes during freezing and thawing in tap water

Concretes made with blends of Type I and Type IA portland cements, 3 x 3 x 11¼-in. prisms frozen and thawed in tap water. Two cycles every 24 hours. Specimens cured 1 day molds, 13 days moistroom, 14 days in air of laboratory and 3 days in water prior to freezing and thawing..

Values in parentheses are the air contents of the fresh concretes determined by the pressure method.

creased resistance to freezing and thawing of concrete containing sand-gravel aggregate. Using air-entraining cements meeting the A.S.T.M. requirement of 18 ± 3 percent air in the mortar test, the sand-gravel aggregate concretes will have air contents ranging from 6 to 12 percent. Air contents in this range will result in only a moderate reduction in strength.

In these tests, expansion, change in dynamic E and change in weight appear to be equally reliable criteria of the resistance to freezing and thawing of these sand-gravel concretes.

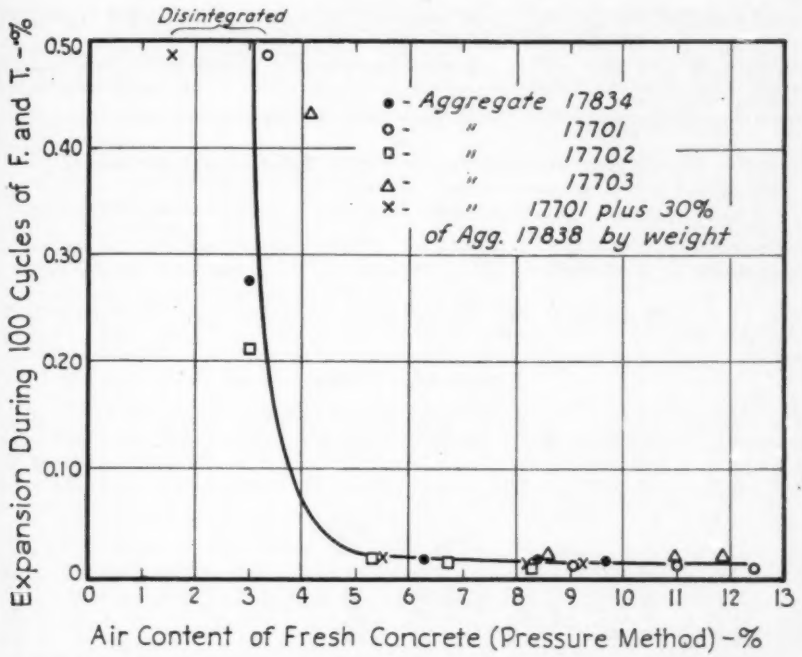

Disintegrated
Expansion During 100 Cycles of F. and T. - %
0.50
0.40
0.30
0.20
0.10
0
0 1 2 3 4 5 6 7 8 9 10 11 12 13
Air Content of Fresh Concrete (Pressure Method) - %
● - Aggregate 17834
○ - " 17701
□ - " 17702
△ - " 17703
× - " 17701 plus 30% of Agg. 17838 by weight

Fig. 7—Relationship between air content of concrete and expansion during 100 cycles of freezing and thawing in tap water

3 x 3 x 11¼-in. prisms frozen and thawed in tap water. Two cycles every 24 hrs. Specimens cured 1 day in molds, 13 days in moist room, 14 days in air of laboratory, and 3 days in water prior to freezing and thawing.

Length changes in moist or dry storage

Concrete prisms, 6 x 6 x 14 in., were made from the same batches as the specimens for strength and durability tests. These specimens were equipped with brass strain gage inserts set to give a 10-in. gage length along the longitudinal axis of both top and bottom surfaces of the prisms as cast. Length measurements made with a Whittemore strain gage are recorded in Table 6 for both moist-cured and air-cured concretes. The concrete prisms are only 90 days old at the time of this writing, and the length changes are small but the results are sufficiently interesting to warrant some comment.

The prisms stored continuously in the moist room showed practically no expansion; in fact, all of the prisms showed a slight shrinkage at 28 days and some showed a slight shrinkage at 90 days. The gain in weight during moist curing was slightly less for the mixes having greater air content.

The prisms stored in the air of the laboratory at 75 F and 50 percent relative humidity showed lower weight loss for the mixes with greater air content. Despite this behavior, the shrinkage for all concretes increased

TABLE 6—LENGTH CHANGES OF MOIST-CURED AND AIR-CURED CONCRETES MADE WITH SAND-GRAVEL AGGREGATES

Specimens: 6 x 6 x 14-in. concrete prisms equipped with brass reference inserts along longitudinal axis of both top and bottom of prism as cast. Change in length over 10-in. gage length measured with a Whittemore strain gage.
Nominal slump of concrete—3 in. Results are the averages of two specimens.

Ref. No.	Air content pressure method, percent	Change* in length and weight for indicated storage, percent							
		Moist storage				Air storage after 7 days moist curing			
		Length		Weight		Length		Weight	
		28 days (+)	90 days (+)	28 days (+)	90 days (+)	28 days (−)	90 days (−)	28 days (−)	90 days (−)
		Aggregate 17834							
1	3.0	−0.001	0.001	0.82	1.17	0.017	0.043	1.74	2.60
3	6.3	−0.002	−0.002	0.96	1.08	0.018	0.047	1.61	2.47
4	8.4	−0.002	0.000	0.72	1.02	0.023	0.052	1.40	2.31
2	9.7	−0.002	−0.002	0.55	0.91	0.024	0.056	1.52	2.12
		Aggregate 17701							
5	3.3	−0.003	0.001	1.31	1.31	0.009	0.029	1.95	3.07
7	9.1	−0.004	−0.002	0.91	1.10	0.016	0.036	1.34	2.37
8	11.0	−0.003	−0.001	1.00	1.06	0.020	0.041	1.24	2.06
6	12.4	−0.006	−0.003	1.21	1.27	0.022	0.045	1.01	1.95
		Aggregate 17702							
9	3.1	−0.003	0.001	1.03	1.20	0.012	0.030	0.97	1.71
11	5.3	−0.002	0.001	1.16	1.16	0.016	0.035	0.75	1.39
12	6.8	−0.002	0.001	1.07	1.13	0.019	0.037	0.89	1.47
10	8.3	−0.004	0.001	0.84	1.08	0.019	0.045	1.13	1.79
		Aggregate 17703							
13	4.2	−0.001	0.001	0.95	1.13	0.013	0.039	1.42	2.54
15	8.6	−0.001	0.002	0.81	0.87	0.015	0.043	1.65	2.74
16	11.0	−0.002	0.001	0.68	0.75	0.017	0.045	1.56	2.55
14	11.9	−0.001	0.002	0.75	0.94	0.020	0.049	1.32	2.07
		Aggregate 17701 plus 30 percent by weight of aggregate 17838							
17	1.5	−0.001	−0.001	0.91	1.03	0.011	0.029	1.87	2.77
19	5.5	−0.001	0.000	0.99	1.04	0.013	0.031	1.40	2.10
20	8.2	−0.007	−0.005	0.89	0.95	0.016	0.035	1.18	1.90
18	9.2	−0.003	−0.001	0.83	0.89	0.016	0.036	1.12	1.83

*Changes for moist-cured concrete based on 1-day measurements (removal from molds).
Changes for air-cured concrete based on 7-day measurements (removal from moist room).

slightly in the series of mixes with the successively larger air contents. Investigations by this laboratory and others[5] show no consistent increase in shrinkage with increase in air content for concretes containing the usual amounts of fine and coarse aggregates and having air contents up to about 6 percent.

CONCLUSIONS

The following conclusions can be drawn from the results of these tests of concretes made with "sand-gravel" aggregates, non-air-entraining and air-entraining portland cements, and blends of these cements:

1. The use of air-entraining portland cement increased plasticity and workability, reduced bleeding and permitted a reduction of ½ to 1¾ gallons of water per sack of cement in a 6½ sack per cu yd mix at a nominal slump of 3 in.

2. The air contents of concretes made with air-entraining portland cement meeting the requirements of A.S.T.M. C175-47T for air content of 1:4 standard sand mortar ranged from about 6 to 12 percent.

3. Both flexural and compressive strength decreased with increased air content. In general, compressive strength decreased 2½ percent and flexural strength 1 percent for each percent increase in air content.

4. In these tests, sand-gravel aggregates used with non-air-entraining portland cement in mixes containing 6½ sacks per cu yd, slump 3 ± ½ in., produced concretes with 3 to 4 percent entrained air. With this amount of air these concretes show relatively low resistance to freezing and thawing, while in concrete containing aggregates graded up to 1½-in. maximum size, 3 to 4 percent entrained air provides a high degree of resistance.

5. With air-entraining cements meeting the A.S.T.M. requirement of 18 ± 3 percent air in the mortar test, the sand-gravel aggregate concretes will have air contents in the range of 6 to 12 percent. These air contents provide adequate resistance to freezing and thawing with only a moderate reduction in strength.

6. The addition of 30 percent of limestone coarse aggregate to the sand-gravel aggregate reduced the amount of air entrained, increased the strength and showed excellent resistance to freezing and thawing for concretes having air content of 5 percent or more.

7. The shrinkage of concrete with sand-gravel aggregates and blends of Type I and IA cements, moist-cured 7 days and then stored in the air of the laboratory at 75 F and 50 percent relative humidity, increased with increase in air content.

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Discussion of a paper by Paul Klieger:

Effect of Entrained Air on Concretes Made with So-Called "Sand-Gravel" Aggregates*

By DUFF A. ABRAMS, M. SPINDEL and AUTHOR

By DUFF A. ABRAMS†

When laboratory studies contradict field experience, we conclude that something is amiss in the laboratory. The author made no claim that air-entraining cements furnish the remedy for the failure of hundreds of miles of "sand-gravel concrete" roads in Kansas, Nebraska, Missouri and Iowa. It is remarkable that the paper did not mention either those roads or the extensive literature on them. It is difficult to understand the purpose of the paper if its teachings are not applicable to these roads that have been investigated during the past 20 years by the local highway departments, by Kansas State College, by the Portland Cement Assn., by the Bureau of Public Roads, and others. The BPR studies have been under way for 12 years but are still incomplete; a progress report by Jackson and Kellermann is in *Proceedings of the Highway Research Board*, 1942. Some of the aggregates in this region have been studied also by the U. S. Bureau of Reclamation.

"Sand-gravel concrete" is only a mortar made of a coarse, and generally poorly-graded sand. The laboratory environment to which the specimens in the paper were exposed did not remotely resemble field conditions that must be met by roads and other structures. The unreasonableness of the freezing and thawing data throws serious doubt on most of the conclusions in the paper.

The author's tests with ordinary cement mixed with sand-gravel "sweetened" by the addition of 30 percent of crushed limestone showed rather early failures in the freezing and thawing tests of prisms. This result is contradicted by many years of experience with concrete roads built of similar mixes. Contrary to the paper, the merits of "sweetening" do not hinge on the use of air-entraining cements. The method

*ACI Journal, Oct. 1948, *Proc.* V. 45, p. 149. Disc. 45-9 is a part of copyrighted Journal of the American Concrete Institute, V. 21, No. 4, Dec. 1949, Part 2, *Proceedings* V. 45.

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was used with complete success long before the special air-entraining cements were invented.

Mortar roads failed due to the violation of engineering principles that were well understood before the roads were built: (a) too much mixing water required by high cement factor and small aggregate size; (b) inadequate curing and (c) shrinkage from evaporation of mixing water. Mortar roads and structures failed from mixes and curing conditions that produced too much air, rather than too little.

FIELD VS LABORATORY EXPOSURE

Mortar mixes studied in the paper are used for structural purposes in a restricted area. Most of the sand-gravels tested originated in western Kansas and Nebraska, and can not be used eleswhere, hence we are justified in concluding that the teachings of the paper apply to that region. This is an elevated plain of low average precipitation (mostly snow), low humidity, extremely hot summers and frequently extremely cold winters. Air temperatures range from 40 below zero to 135 F, a range of 175 F. This was the middle of the famous dust bowl of 1934. Repeated freezing and thawing of mortar at early ages while saturated with water is not a typical field condition.

Freezing and thawing temperatures used by the author were not reported. The points in Fig. 6 show that the prisms were removed from water after 5, 10, 25, 50, 75, 100, 125, . . . cycles, for measurements that required considerable time. It is probable that the low-air prisms retained their absorbed water, but the high-air prisms allowed the water to run out. When the latter were again plunged into water, a large quantity of air was entrapped which acted as a cushion against the otherwise destructive expansion from the formation of ice. The low-air prisms, deprived of the advantage of this entrapped air, were quickly destroyed by repeated freezing and thawing, while those of high air content survived.

This interpretation is supported by data in the paper: (a) Fig. 6 shows that when entrained air in fresh mortar exceeds 4.2 percent, the quantity of air has no appreciable effect on expansion, dynamic modulus, or change in weight; (b) Fig. 7 shows that at 100 cycles all prisms with more than 4.2 percent of air gave negligible expansion, while those with less than 4.2 percent of air failed.

It will be shown that the roads that failed had about 5 percent of air as fresh mortar, and about 15 percent as hardened mortar, hence the criterion for durable concrete set up by the author was met. This throws the gravest doubts on both the applicability of the freezing and thawing tests, and on the interpretation of the author of the role of special air-entraining portland cements.

AIR CONTENT OF FRESH MORTAR

Probably the most important deduction is in Conclusion 5:

"These air contents (6 to 12 percent) provide adequate resistance to freezing and thawing with only moderate reduction in strength."

The air content refers to fresh mortar, and not to the hardened state when the mortar is exposed to freezing. The above conclusion makes it desirable to inquire into the air content of the mortar roads. The briefest consideration shows that the air content of fresh mortar has little relation to the air content of hardened mortar. The paper did not give data on road mixes and the test mixes departed materially from those used in the field, but fortunately, we find data in the paper by Jackson and Kellermann. The differences between the author's mixes and the road mixes, using Platte River aggregates, may be shown:

	Klieger	Jackson and Kellermann
Maximum size of aggregate	⅜ in.	No. 4
Fineness modulus	4.17	3.75
Cement, sacks per cu yd	6.5	7.55
W/C, by weight	0.4	0.44
Slump, in.	3	1

For Type I cement and Platte River aggregate, Jackson and Kellermann gave:

Cement, sacks	Solids, percent	Data from Table
7.38	77.6	4
7.70	76.6	
7.35	77.4	6
7.75	77.3	
7.55	77.2	

We can compute the volume of mixing water and estimate the air content of the fresh road mortar.

Mixing water in BPR tests:

$7.55 \times 94 \times 0.44 = 312$ lb per cu yd, or $\dfrac{312}{62.4 \times 27} = 18.5$ percent by volume.

Air in fresh mortar:

$100.0 - (77.2 + 18.5) = 4.3$ percent.

The above tests agree with the paper in showing that a higher cement content is accompanied by increased air.

While the BPR tests were intended to simulate field mixes, with 7.75 sacks of cement, Jackson and Kellermann state of the sand-gravels:

" . . . the fine grading has in some cases necessitated the use of cement contents as high as 7.5 to 8 sacks per cu yd"

or 15 to 23 percent more cement that the mixes used by Mr. Klieger. The BPR tests showed 4.3 percent of air with 7.55 sacks of cement; with 7.75 sacks, the average for the earlier roads, the air content of fresh mortar would be about 5 percent.

AIR CONTENT OF HARDENED ROAD MORTAR

In dealing solely with the air content of fresh mortar, the paper avoided the fact that the air content of fresh mortar is a fleeting condition that has little relation to the air content of mortar when in service.

Mortar roads were water-cured for 7 days, then allowed to dry out. In water-curing ordinary cements for 7 days the fixed water amounts to 0.14 by weight of the cement, hence is nonevaporable. The air content of hardened mortar would then be:

Air in fresh mortar: 5 percent.
Mixing water: $7.75 \times 94 \times 0.44 = 321$ lb per cu yd.
Water fixed at 7 days: $7.75 \times 94 \times 0.14 = 102$ lb per cu yd.

Evaporable water: $321 - 102 = 219$, or $\dfrac{219}{62.4 \times 27} = 13$ percent by volume.

Absorbed water—The desiccating atmosphere quickly removes water absorbed by aggregates, or about 2 percent by volume.

Hydration solids—In moist curing, water becomes fixed by cement hydration and forms solid hydration products. This reduces the quantity of air, as computed above. For the usual cement the absolute volume is increased 0.36 by 7 days moist curing. The absolute volume of cement is about 0.48 of loose volume, hence in hardened road mortar we have hydration solids:

$7.75 \times 0.48 \times 0.36 = 1.34$ cu ft per cu yd, or $1.34/27 = 5$ percent, which may be considered as a negative percentage of air.

Total air content—Near exposed surfaces, all evaporable and all absorbed water would be lost in a few weeks. By the time the road is exposed to freezing the air content would be about:

$$5 + 13 + 2 - 5 = 15 \text{ percent.}$$

AIR CONTENT OF HARDENED LABORATORY MORTAR

If the above computed air content of road mortar seems excessive, let us present parallel computations from the paper, using an air-entraining cement and the same Platte River sand-gravel aggregate.

Air in fresh mortar (mix 6): 12.4 percent.
Mixing water: $6.4 \times 4.6 \times 8.3 = 244$ lb per cu yd.

Water fixed at 7 days—The exact amount of water fixed by air-entraining cement has not been reported; evidently it is a little less than for Type I cements; we assume 0.12:

$6.4 \times 94 \times 0.12 = 72$ lb per cu yd.

Evaporable water after 7 days curing:

$$244 - 72 = 172 \text{ lb, or } \frac{172}{62.4 \times 27} = 10.2 \text{ percent.}$$

Absorbed water: 2 percent.

Hydration solids: $6.4 \times 0.48 \times 0.31 = 0.95$ cu ft, or $0.95/27 = 3.5$ percent.

Total air content—When this mortar was exposed to its first freeze, its air content was about: $12.4 + 10.2 + 2.0 - 3.5 = 21$ percent.

The actual air content of 21 percent should be contrasted with the 12.4 percent which the author used throughout the paper for this mix.

The paper showed that after moist curing for 7 days, followed by exposure to laboratory air, nearly ½ of the evaporable water was lost at 90 days. It is a foregone conclusion that, under the drying conditions of western Kansas and Nebraska, most of the evaporable water would be lost during the first few weeks after the end of the curing period.

TESTS NOT APPLICABLE TO MORTAR ROADS

Mr. Klieger claims that a fresh mortar air content of 6 percent "provides adequate resistance to freezing and thawing" and his tests show that 5 percent is sufficient. But we saw that the air content of the fresh mortar in the roads was about 5 percent. This leaves no margin whatever between permanence and failure, and throws grave doubts on the validity of the freezing and thawing tests, and on all conclusions that the author based on his tests.

It is an odd notion that, after hundreds of miles of mortar roads containing about 15 percent of air had failed, permanent roads will result by increasing air content to 21 percent. Mortar containing 21 percent of air is not a satisfactory substitute for *good concrete.*

STRENGTH OF AIR-ENTRAINING CEMENT MORTARS

For 200 years compressive strength has been used as a criterion of the quality of mortar and concrete. Strengths are of little value unless they are made under standardized conditions. For 30 years the 6 x 12-in. cylinder has been almost universally used. The author adopted the "modified cube"—a specimen little used, the values from which are understood by few. Even the name is misleading, since it is not a cube, but is a bearing stress applied to part of a beam. The lower height-diameter ratio and the restraint of overhanging ends gives strengths about 1½ times that of the cylinder.

We summarize the strengths of four air-entraining cement mortars that the author found to "provide adequate resistance to freezing and thawing with only moderate reduction in strength":

Sand-gravel No.	Cement sacks	Entrained air, percent	Fluid-cement ratio (F/C)	Modified cubes 28-day strength, psi
17834	6.6	9.7	1.07	4280
17701	6.4	12.4	1.21	3910
17702	6.5	8.3	1.03	4910
17703	6.4	11.9	1.21	3900
Average	6.5	10.6	1.13	4250

Except for column 4, data are from the paper. The writer's experiments that led to the water-cement ratio method of concrete mix design clearly showed that strength and most of the other properties of concrete are functions of W/C, and that the loss of strength and resistance due to increasing W/C resulted from the *dispersion of the cement particles.* Whether dispersion was due to water or air made no difference. Table 5 of the paper gave "void-cement ratios" of the fresh mortar; voids, V, being the sum of water and air, and C the absolute volume of cement. Absolute volume of cement is about ½ the loose volume, hence if we divide the author's V/C values by 2, we transfer the values to the more familiar form. The combined effect of water and air, in the above table is expressed as F/C; this avoids confusion with W/C.

In terms of standard cylinders the 28-day strength of the mortars averaged 4250/1.5 = 2840 psi, and the minimum was 2600 psi. The 1940 Report of the Joint Committee specified a minimum strength of 4000 psi for concrete bearing slabs. To meet this requirement an average strength of about 5000 psi is necessary. This leads to two conclusions:

(1) The author's "moderate reduction in strength" amounted to about $\frac{5000 - 2840}{5000}$ = 43 percent.

(2) The author approves building mortar roads with the equivalent of W/C 1.21, by volume, and a strength equivalent to 2600 psi, in terms of standard cylinders.

DRYING SHRINKAGE OF CONCRETE

Restrained concrete develops destructive shrinkage stresses when exposed to the comparatively mild drying conditions of the testing laboratory. The effects of this loss of water have been well understood since the Portland Cement Assn. published Series 216, in 1929. We quote:

In the second stage (after hardening) as in the first, shrinkage is the result of loss of water from concrete. The shrinkage is also a function of the quantity of cement in the mix. The quantity of water lost for any given condition of curing is a function of the quantity of mixing water.*

**Proceedings*, PCA Spring Meeting, 1929.

The important facts are that shrinkage is a function of the quantity of both cement and water. The PCA tests indicate that for mixes used in the mortar roads, shrinkage would be about 1½ in. per 100 ft, after moist curing for 7 days, followed by exposure to laboratory air for 6 months. These tests have been publicized during the past 13 years in *Concrete Manual*, U. S. Bureau of Reclamation, 1936 to 1942.

The above PCA tests are confirmed by those in the paper. Beams 6 x 6 x 14 in., made of air-entraining cements, moist cured for 7 days, then in laboratory air until 90 days, contracted on the average 0.00049, which may be compared with 0.00130 in the earlier tests at 6 months.

The paper did not report modulus of elasticity (E); but we shall not be far astray if we assume $E = 1000$ times cylinder strength. The "modified cubes" averaged 4120 psi at 28 days hence cylinder strength will be about 2750 psi and E about 2,750,000 psi. The tensile stress developed in a restrained member would be about $0.00049 \times 2{,}750{,}000 = 1350$ psi. The conditions at 90 days would not be materially different. If partially restrained, as in a road slab, the prisms probably would have cracked long before they were 90 days old.

CORRECTIONS FOR PAPER

Various statements in the paper on the makeup of aggregates in which sand-gravel was mixed with crushed limestone are confusing and(or) misleading. If the sand-gravel was the original 100 percent, the sieve analysis in Table 3 is erroneous, and the fineness modulus should be 4.90 instead of 4.81. If the combined sieve analysis is correct, 26 percent of sand was used instead of 30 percent as stated in paper. If the 30 percent of crushed stone was "by total weight of aggregate," as stated on p. 159, Table 4 is in error and the combined fineness modulus is 5.10.

The legend of Fig. 5, "Relation between void-cement ratio and strength of concrete . . . ," is erroneous. The V-values are for fresh mortar and do not represent the conditions at 28 days when the tests were made. At 28 days, three corrections are necessary; two of them lower V, the other increases C: (1) bleeding caused the loss of as much as 15 percent of mixing water; (2) solids gained by cement hydration reduce V-values and (3) bleeding materially increased cement factor C.

All corrections tend to reduce V/C, hence all curves in Fig. 5 must be shifted considerably to the left.

A MISLEADING CONCLUSION

The author's Conclusion 1:

> The use of air-entraining portland cement increased plasticity and workability, reduced bleeding and permitted a reduction of ½ to 1¾ gal. of water per sack of cement in a 6½ sack per cu yd mix at a nominal slump of 3 in.

is likely to mislead the reader who does not fully analyze the significance of the "voids-cement ratio," based on absolute volumes. The writer raises three objections to the quoted conclusion: (1) the reduction of 1¾ gal. was based on two tests with "sweetened" aggregates (17 and 18) both of which seem to be erratic; (2) with sand-gravel aggregates (the field in which the results seem to be particularly applicable) reduction in mixing water with air-entraining cement ranged from 0.7 to 1.1 gal. per sack and (3) the apparent reduction in mixing water by the use of air-entraining cement was largely fictitious and accomplished no useful purpose, since the water was replaced by an equal volume of air which had the same effect on strength as the water replaced.

The tests of air-entraining cement raised many new questions. They did not answer the old question: How to build permanent roads with sand-gravel aggregates?

By M. SPINDEL*

It is not at all surprising that concretes made with sand-gravel aggregates having a maximum particle size of ⅜ in., instead of ¾ to 1½ in., were found to contain 3 to 4 percent air voids, even when ordinary non-air-entraining portland cement Type I was used. The important question is why the effect of 3 to 4 percent of air entrained in usual concretes by means of air-entraining agents are sufficient to secure resistance to frost action, as tested by the methods shown in the paper, while the same amount of air obtained without an air-entraining agent had no beneficial effect on the resistance to frost action. To secure such resistance, it was necessary to entrain at least another 3 to 4 percent of air by means of an air-entraining agent added to the cement.

The above seems to confirm the opinion given in various papers and discussions, especially in a paper by M. Spindel and R. T. Quinn.† Referring to the writer's discussion in the ACI JOURNAL‡ that both the water-cement ratio and the air-cement ratio had to be limited to secure satisfactory results, it was repeated in the above paper that "apart from the percentage of air, the size, shape and distribution of air pores was highly important for success or failure." In the discussion, the question of why and how the size, shape and distribution of air pores should be determined, the writer's answer was as follows:

The air voids were very different when concrete was made too dry or too wet or workable with the lowest water-cement ratio. The shape, size and distribution of air voids could be tested by microscopic examination and by appropriate absorption tests; the latter gave the so called Saturation Coefficient which indicated the resistance to frost of concretes and natural stones alike.

*London, England.

†Spindel, M. and Quinn, R. T., "Improvements on Portland Cement and Concrete—Past, Present and Future," *Transactions of the Society of Engineers* (London), Dec. 1945.

‡Discussion of "The Function of Entrained Air in Concrete," ACI JOURNAL, Nov. 1943 (Supplement), *Proc.* V. 39, p. 544-1.

Since 1943 much valuable research has been done in the above direction and the results were published mainly in the ACI JOURNAL, especially in the paper "The Camera Lucida Method for Measuring Air Voids in Hardened Concrete," by George I. Verbeck* and recently in the paper under discussion.

Hardened concrete should be tested microscopically much more carefully than to examine only the sizes, numbers and distribution of air pores by the camera lucida method. The microscopic examination of hardened concretes will have to be done in the same way as the highly developed microscopic examination of cement clinkers in order to show both the composition of the hardened cement paste and the air voids. The latter should not be filled with a colored substance to make the air voids visible as for the camera lucida method, but on the contrary, it will be very useful to examine the whole surfaces of the air voids regarding the various crystals, etc., which are usually deposited there and, without doubt, have some significance with regard to the tests for resistance to frost.

It will be especially necessary to distinguish the various types of air voids; those which are the result of wrong proportioning and grading or insufficient compacting of the concrete are different from those obtained by entrained air which improve the resistance to frost. Finally, those air pores which result from the evaporation of the mixing water, which could not be bound chemically by the hardening of the cement, must not be overlooked. These last mentioned air voids, if exceeding certain limits, have the worst effect on the durability of mortars and concretes. The examination and measurement of the various types of air voids has to be made along with specially designed absorption tests, showing the absorption at various times and under various conditions of immersion in water, to find the "saturation coefficient."

In the discussion of a symposium "Entrained Air in Concrete"† the writer pointed out that "at any rate it might be of great advantage if all concrete constituents were given also by absolute volume per unit of fresh concrete." This is also true with regard to the instructive tables and figures of the paper under discussion. Since 1928 the writer pointed out in many publications, one of which was his "Report on Special Cements" submitted to the Second International Congress on Large Dams, Washington, D. C., 1936, that concrete must be considered as composed of the four components: cement, aggregate, water and air, which four components had to be shown in their absolute volumes in the four matter parallelogram which gives not only the percentages but also all relations of the components to each other and their effect on all properties of concrete depending on the various relations. It is a pity that the author did not make use of it.

*See ACI JOURNAL, May 1947, *Proc.* V. 43, p. 1025.
†Discussion of "Entrained Air in Concrete," ACI JOURNAL, Dec. 1946, Part 2, *Proc.* V. 42, p. 700-1.

What useful information could be easily obtained from the four matter parallelogram in this case?

1. All concrete mixes having the same cement factor would be represented by points on the same straight line parallel to the vertical axis, and from each water-cement ratio and volume weight of the various concrete mixes the exact position of the point and another point which is its horizontal projection could be determined without any further calculations, thus giving the percentages in absolute volume of cement, aggregate, water and air. These percentages should be compared with those obtained by the microscopic examination in Fig. 2.

2. For every concrete mix, which is represented by two points, *i.e.* the vertical and horizontal projection, in the four matter parallelogram, the relation between the percentage of entrained air and the reduction of the percentage of water by absolute volume, or even of the reduction of the water-cement ratio, could be seen without any further calculations as in the writer's discussion of the symposium on entrained air.

3. The relation between the air and cement paste, *i.e.*, cement plus water in absolute volumes, which is more significant than the relation between air and the whole volume of concrete, which means the percentage of air, could be seen from the parallelogram and compared with the microscopic image shown in Fig. 2 of the paper. Further the relation between cement paste plus air and the whole volume of the concrete could be seen. This relation has been assumed to be 25 percent in the average for concretes made with the usual aggregates in the paper by George I. Verbeck but is, of course, different for various cement factors and also for aggregates of a special nature.

4. For every point representing a concrete mix in the parallelogram, all the usual relationships, such as the water-cement or cement-water ratio, the voids-cement or the cement-voids ratio, and finally the cement-space or space-cement ratio could be seen without any further calculations, together with the expected compressive strengths as to the various formulas based on these relations, such as for example the most used formula by Talbot and Richart. It is, of course, important to compare the expected compressive strength with those actually obtained by the tests carried out.

Apart from the percentage of air in the fresh concrete, there is a high percentage of air voids resulting from the evaporation of a considerable part of mixing water during the hardening of concrete. These air voids are different in size, shape and distribution from the air voids usually obtained by entrained air and, without doubt are very harmful regarding durability if exceeding certain limits. This has been shown recently in several articles in European periodicals by Dr. Amman. The main point of the above publications was to show the advantages of entrained air and the disadvantages of the air pores resulting from the evaporation of the mixing water, which were partly eliminated by the air pores obtained by entrained air and by the reduction of the water-cement ratio. The conversion of mixing water into air voids by evaporation has been shown too, by the writer on the four matter parallelogram.*

The various tables and figures of the paper show that the resistance to frost was very low with air voids of 3 to 4 percent obtained with the ordinary non-air-entraining cement and that a further 3 to 6 percent of air entrained by air-entraining cements was necessary to secure resistance to frost. Two questions arise:

**Beton u Eisen*, 1931, No. 1 and 2.

(A) Is it suitable to mix ordinary cement Type I with air-entraining cement Type IA in various proportions to obtain the percentage of entrained air considered to be the most suitable for the job in question?

(B) Whether a concrete having an air content of say 11.0 percent or 12.4 percent of the whole concrete and much more percentage of the cement paste only, as shown in Fig. 2, can be considered as a reliable material for structural purposes, although the test for resistance to frost gave highly satisfactory results. The writer is still of the opinion that the air content always should be limited in some way to avoid failures.

AUTHOR'S CLOSURE

Sand-gravel aggregate concretes similar to those described by the author have been used in pavements and structures in Kansas, Nebraska, Iowa and western Missouri. It is well recognized that some of these aggregates have a poor service record as described in previous papers by Jackson and Kellermann,[1] and Scholer and Gibson.[2] The varying performance of aggregates from different sources is not primarily dependent on the grading characteristics of these materials but appears to be caused by a cement-aggregate reaction and the aggregates that show poor performance can be improved by "sweetening" with coarse limestone aggregate as shown by Scholer and Gibson.[2]

It was not within the scope of the paper under discussion to study this cement-aggregate reaction. The purpose of the present paper was to show whether concretes made with sand-gravel aggregates could be improved by air entrainment.

Mr. Abrams discusses the air content of air-dried concretes at considerable length. It is recognized that concrete loses water on drying and that the space occupied by water will be filled with air when the concrete dries. It is recognized also that the evaporable water is the freezable water in the concrete. Consequently air-dried concrete that contains little or no freezable water will not be damaged by alternate cycles of freezing and thawing. The freezing-and-thawing tests described in the paper under discussion were made with saturated concrete. Pavements and structures exposed to the weather often are subject to alternate cycles of freezing and thawing while saturated. The air content of the concrete in that condition would be very nearly the same as that of the freshly mixed concrete. The results clearly indicate that entrained air greatly increased the resistance to freezing and thawing.

The writer is not familiar with the source of information used to justify the statement that the compressive strengths of modified 6-in. cubes are 1.5 times that of 6 x 12-in. cylinders. H. F. Gonnerman[3] reported data from different sources which showed that 6-in. cube strengths were about 15 percent higher than those obtained with 6 x 12-in. cylinders. On the basis of this relationship, the compressive strengths of 6 x 12-in. cylinders and the modulus of elasticity of the concretes

described in the paper would be very much higher than the calculated values given in Mr. Abrams' discussion.

Mr. Abrams' statement that with sand-gravel aggregates the reduction in mixing water with air-entraining cement ranged from 0.7 to 1.1 gal. per sack is incorrect. The data in Table 4 of the paper show that for the sand-gravel aggregates the reduction in net water-cement ratio ranged from 0.5 to 1.7 gal. per sack. With the "sweetened" aggregate the reduction in water-cement ratio was 1.7 gal. per sack and the results do not seem to be erratic since in four mixes (No. 17, 19, 20, 18) there was a progressive decrease in water-cement ratio with increasing air contents.

The writer is indebted to Mr. Abrams for pointing out the error in the combined grading of the sweetened mix. The limestone was used in an amount equal to 30 percent by weight of total aggregate. The corrected grading and fineness modulus are as follows:

Lot No.	Source	Percent retained—Cumulative								
		1–in.	¾–in.	⅜–in.	Sieve No.					
					4	8	14	28	48	100
17701 + 30 percent by wt. of 17833	S. Bend, Neb. + Chicago Limestone	0	8	28	41	57	83	95	98	99
		(Fineness modulus = 5.09)								

Mr. Spindel raises some questions that cannot be answered at this time. Studies of the size and distribution of the air voids, of the saturation coefficient, and their respective effects on resistance to freezing and thawing, are under way in this and other laboratories. In the interest of brevity, a tabulation of the absolute volumes of the various constituents of the concrete was not included in the paper. However, the necessary data for the calculation of these absolute volumes are shown in the various tables of the report.

Blends of Type I and Type IA cements have been used extensively in laboratory tests and on construction projects. The results obtained have compared favorably with those obtained using individual cements of different air-entraining capacity. Some limit on the air content of concrete seems desirable to minimize the reduction in strength associated with air entrainment.

REFERENCES

1. Jackson, F. H. and Kellermann, W. F., "Volume Changes in Sand-Gravel Concrete," *Proceedings*, Highway Research Board, 1942, p. 252.

2. Scholer, C. H. and Gibson, W. E., "Effect of Various Coarse Aggregates Upon the Cement-Aggregate Reaction," ACI JOURNAL, June 1948, *Proc.* V. 44, p. 1009.

3. Gonnerman, H. F., "Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete," *Proceedings*, A.S.T.M., V. 25, Part 2, 1925.

Mr. Petersen's paper, based on National Bureau of Standards studies, reports on burned shale and expanded slag concrete of relatively low weight and thermal conductivity and excellent resistance to rain penetration.

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture*

By P. H. PETERSEN†

SYNOPSIS

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration, and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

INTRODUCTION

An investigation was conducted at the National Bureau of Standards to determine the physical properties of several lightweight aggregate concretes in anticipation of their use in the monolithic construction of buildings. These lightweight aggregates of burned shale or expanded slag incorporated features which tend to improve heat insulation more than the aggregates used for dense concrete. However, if the cement content is low (3 sacks per cu yd) workability is so poor that placement of the concrete is extremely difficult. Instead of flowing into the forms, the mixture is harsh and the mixing water tends to settle, taking some cement with it. This fault may be overcome by using an admixture to entrain 20 to 25 percent air in the concrete. When the cement content is increased to 10 or 11 sacks per cu yd of concrete, there is no need for entraining air to increase workability.

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For tests described in this report, burned shale and expanded slag were each used as aggregates in three grades of concrete ranging upward from a strength of 500 psi. An air-entraining agent was added to the two leanest mixtures for each aggregate but was omitted in the concretes which were richest in cement.

The properties covered in this investigation include: compressive, flexural and bond strengths; resistance to heat transfer; water penetration by capillarity; and rain penetration. Values were determined also for shrinkage, for the coefficients of thermal expansion, and for length changes due to wetting and drying.

MATERIALS

Cement

The cement was a portland cement conforming to the requirements of federal specification SS-C-191b, Amendment 1.

Aggregates

The burned shale was a lightweight cellular aggregate made by burning a certain clay and shale to incipient fusion. The material was obtained in two sizes, coarse and fine. Sieve analyses, absorptions (surface dry), bulk specific gravities, and unit weights are given in Table 1.

The expanded slag was a lightweight cellular aggregate made by agitating molten hot slag as it comes from the blast furnace and cooling it rapidly in an atmosphere of steam. This material also was furnished in coarse and fine sizes. The physical properties are given in Table 1.

TABLE 1—AGGREGATE DATA

Aggregate	Burned shale		Expanded slag	
Grade	Coarse	Fine	Coarse	Fine
Sieve analysis: Amount passing U. S. standard sieve, percent				
½ in.	100		100	
⅜ in.	88		98	
No. 4	20	100	34	100
No. 8	5	97	13	96
No. 16	4	64	11	76
No. 30	3	38	10	49
No. 50	3	25	10	30
No. 100	3	17	9	21
No. 200	2	12	7	14
Absorption (surface dry)[a], percent 48 hr at 70 F	15.8	10.9	36.8	19.0
Bulk specific gravity (oven dry)[a]	1.09	1.54	1.09	1.46
Weight, loose dry, lb/cu ft	37	45	40	57

[a]Absorption and bulk specific gravity determined by A.S.T.M. methods of test C127-42 and C128-42.

Air-entraining agent

The admixture was a commercial product used for entraining air in concrete.

CONCRETE USED IN TEST SPECIMENS

Batching and mixing

The cement and lightweight aggregates of burned shale and expanded slag were proportioned by trial batches to produce concretes ranging from a low to a high cement content. Three mixtures of each were selected to produce a low-, a medium- and a high-strength concrete.

The burned shale aggregate, in the original sacks, was completely immersed in water for 48 hours and then was allowed to drain, while still sacked, for 24 hours. The moisture content was determined and the amount of free water was included as mixing water in the proportioning of the materials. The expanded slag aggregate was used in the dry condition as recommended by producers of this material.

TABLE 2—PROPORTIONS OF INGREDIENTS, DENSITY AND ABSORPTION OF CONCRETES

Concrete mix No.	1	2	3	4	5	6
Aggregate	Burned shale	Burned shale	Burned shale	Expanded slag	Expanded slag	Expanded slag
Cement content, sacks per cu yd	3.0	5.4	9.9	4.4	6.9	11.0
Air entraining agent, percent by weight of cement	0.50	0.25	none	0.50	0.25	none
Proportions, by loose volume, dry:						
Portland cement	1.0	1.0	1.0	1.0	1.0	1.0
Fine aggregate	6.2	3.2	1.4	2.7	1.5	0.8
Coarse aggregate	3.0	2.0	1.5	2.2	1.6	1.2
Water-cement ratio by wt[a]	1.36	0.82	0.52	—	—	—
Slump, in.	2.5	4.3	6.4	4.0	6.0	5.6
Cement-voids[b] ratio by absolute volume	0.122	0.247	0.611	0.174	0.303	0.652
Entrained air, percent by volume[c]	20.8	13.9	[d]	(24)	(18)	[d]
Weight of fresh concrete, lb per cu ft	74.0	83.4	101.2	72.9	81.3	102.8
Weight of hardened concrete, lb per cu ft	60.3	71.4	88.2	65.1	72.8	96.2
Water absorption, (24 hours at 70 F), percent of dry weight	23.1	14.0	10.0	21.4	14.6	10.0
Capillary rise of water, in.						
At 1 hour	1.9	1.3	1.0	0.9	1.1	0.9
At 7 days	8.5	4.7	4.0	6.3	4.6	4.8

[a]Amount of water includes surface moisture in wet burned shale aggregate, but data are not available for the dry expanded slag aggregate concretes.
[b]Voids are entrained air and mixing water.
[c]Values in parentheses are estimated.
[d]Negligible.

The materials were proportioned by weight and the concrete was mixed in a standard 6-cu ft rotary drum mixer. The air-entraining agent, when used, was introduced with the water and the time of mixing was in all cases about 5 minutes, since trial mixes indicated that this was the minimum interval needed to secure complete air entrainment.

All specimens were hand puddled. The forms were stripped at one day, the specimens cured with wet burlap 7 or 28 days as required and then stored in the laboratory until tested.

Table 2 lists the data obtained on the fresh concrete and the weight and absorption of the dry concrete.

Properties of the dry concrete

The weight of dry concrete was obtained after drying the specimens to constant weight in a chamber maintained at 122 F (50 C) where the humidity was lowered to approximately 20 to 30 percent by means of pans of calcium chloride. The cement-voids ratio shown in Table 2 is based on the absolute volume of all the ingredients, the voids including entrained air as well as all water other than that absorbed in the aggregate. Water absorption was determined on oven-dried specimens (220 F) by immersing them in water at 70 F for 24 hours.

Water penetration due to capillarity was determined by placing three specimens of each concrete in water $\frac{1}{8}$ to $\frac{1}{4}$ in. deep. The specimens were 4 x 4 in. in cross section and about 12 in. high. Observations were made at 1 hour and at 7 days to determine the rise of water due to capillarity (data in Table 2).

STRENGTH TEST RESULTS

Compression

The specimens for compressive strength tests included 6 x 12-in. cylinders and some wallettes 8 x 30 x 30 in. The wallettes were used primarily to obtain values of shrinkage and were then tested in compression at the termination of those observations.

Values for the secant modulus of elasticity were obtained in cylinders wet-cured for 28 days by means of Tuckerman extensometers with a 6-in. gage length. The moduli were determined at stresses of 200, 800 and 2,000 psi respectively for the three grades of concrete. These values and the compressive strength values are given in Table 3.

Flexure

Flexural strength tests were made on bars 4 x 4 x 30 in. at 28 days. The bars were supported as a simple beam on a span of 24 in., and were loaded at the center with the screeded side of the concrete in compression. The computed values for the modulus of rupture are included in Table 3.

TABLE 3—COMPRESSIVE AND FLEXURAL STRENGTHS OF CONCRETE SPECIMENS[a]

Concrete mix No.	1	2	3	4	5	6
Compressive strength, psi, Cylinders, 6 x 12 in.						
7 days wet-cured	230	1110	4250	310	960	2560
28 days wet-cured	480	1730	5720	590	1510	3380
28 days, 7 wet- and 21 air-cured	530	2240	6090	770	1660	3540
2 years, 7 days wet-cured	650	2590	5850	860	1570	3100
Wallettes, 30 x 30 x 8 in. 2 years, 7 days wet-cured	1030	2970	5380	810	1590	3150
Secant modulus of elasticity, psi, 6 x 12-in. cylinder, 28 days wet-cured,						
At 200 psi	720,000			615,000		
At 800 psi		1,240,000			1,050,000	
At 2000 psi			2,170,000			2,000,000
Modulus of rupture in flexure, psi, 4 x 4-in. bars, 28 days, 7 wet- and 21 air-cured	153	398	690	183	333	656

[a]All values are the average for 3 specimens except in the case of wallettes of which there was only one specimen each.

Bond

Reinforced beams were made of the two strongest grades of each concrete with two types of bar represented. Bond strengths after 28 days of wet curing were determined for lengths of embedment of 10, 20 and 30 in. by making and testing beams of different lengths. A sketch of the beams is shown in Fig. 1.

The two bars represented are shown in Fig. 2 and have nominal diameters of 1 in. The deformations on bar A are of the double helix pattern, reversing direction on the two faces of the bar. The deformations on bar B are of the diamond-patterned type. The tensile properties determined for each bar are given in Table 4.

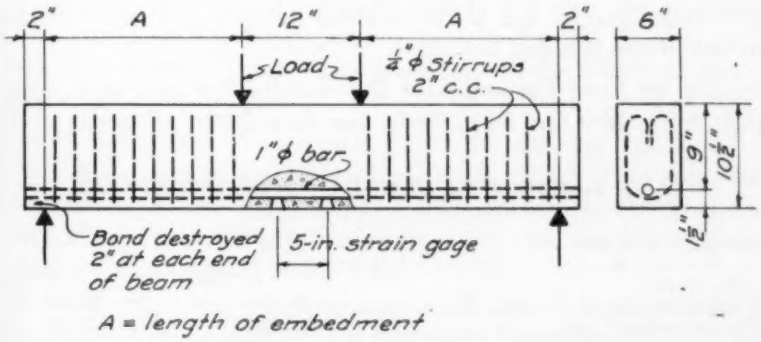


Fig. 1—Sketch of beams for bond-strength tests

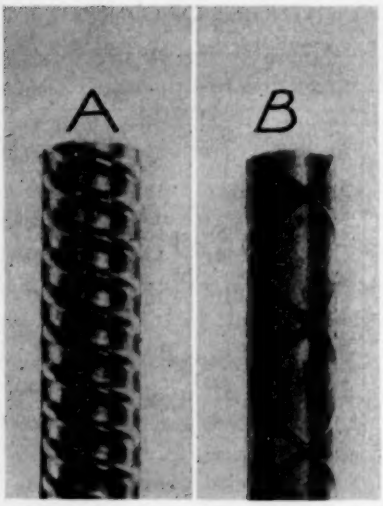


Fig. 2—Types of bars for bond-strength tests

Most of the specimens exhibited bond failures, but in some the reinforcing bars yielded in tension. Where the latter occurred, bond stress values were based on the load at which yielding of the reinforcement occurred. Whittemore strain gage readings on a 5-in. gage length were made on the steel at the center of the beam through holes in the concrete. Slip at the ends of the bar was measured by using 0.0001-in. dial indicators fastened to two lugs embedded in the concrete, the dial stem being in contact with the end of the bar.

The computed bond stresses at an initial slip of 0.001 in. and the bond strengths are given in Table 5.

THERMAL PROPERTIES

Heat transfer tests

Two types of specimens of each concrete were made for the heat-transfer tests, one for use in the shielded hot box and one to be tested by means of the guarded hot plate.

Shielded hot box—The walls for the shielded hot box tests were 100 in. high, 8 in. thick and 56 in. wide, and were tested at an age of 18 to

TABLE 4—TENSILE PROPERTIES OF REINFORCING BARS

Designation of bar	Bar A	Bar B
Area, sq in.	0.796	0.788
Yield point, psi	60,400	52,000
Tensile strength, psi	94,200	92,300
Elongation percent in 8 in.	18.8	23.1

TABLE 5—BOND BETWEEN CONCRETE AND REINFORCING BARS

Concrete mix No.	2	3	5	6
Compressive strength, 28 days wet, psi	1730	5720	1510	3380
Bond stress (psi) at 0.001 in. slip:				
Bar A				
10-in. embedment	320	647	283	440
20-in. embedment	425	600	320	500
30-in. embedment	420	520[a]	307	520[a]
Bar B				
10-in. embedment	182	528	212	430
20-in. embedment	175	425	165	340
30-in. embedment	157	450[a]	180	450[a]
Bond strength, psi				
Bar A				
10-in. embedment	341	820	318	440
20-in. embedment	506	790[a]	374	610
30-in. embedment	520[a]	520[a]	328	520[a]
Bar B				
10-in. embedment	310	619	231	530
20-in. embedment	385	680[a]	221	544
30-in. embedment	267	450[a]	252	450[a]

[a]Bond stress at yielding of bars.

24 months. They had been wet-cured 7 days and then exposed to the air of the laboratory the remainder of the period.

The heat transfer section of the National Bureau of Standards conducted the tests in a shielded hot-box apparatus designed for the purpose. During the test, heat flowed from the metering and shield boxes (which were heated electrically) through the wall to the cold box which was cooled by a refrigerating machine. The electric energy supplied to the metering box and measured by a watt-hour meter was closely equivalent to the heat energy transferred through the portion of the wall covered by the metering box. The temperature in both the metering and shield boxes was kept at 70 F; the temperature in the cold box was kept at 0 F, making the mean temperature approximately 35 F. Air temperatures and surface temperatures on both sides of the wall were measured by means of thermocouples.

Guarded hot plate—The specimens for the guarded hot plate tests were 8 x 8 in. square and about 1 in. thick. They were cut from an 8-in. wall sample with one 8-in. dimension parallel to the thickness of the wall, the other parallel to the height of the wall, and the 1-in. dimension parallel to the width of the wall.

The apparatus consisted of an electrically heated flat hot plate, 8 in. square, flanked on each side by a cold plate that was cooled by a flow of water at constant temperature. Two like specimens of each concrete

TABLE 6—HEAT TRANSFER COEFFICIENTS

Concrete mix No.	1		2	3	4	5	6
Shielded hot box data:							
Thickness of walls, in.	8.05		8.06	8.02	8.00	8.00	8.00
Corrected thermal transmission[a], U	0.26		0.32	0.41	0.24	0.30	0.38
Thermal conductivity[b], k	2.60		3.44	4.88	2.41	3.13	4.34
Guarded hot plate data:							
Thickness of specimen, in.	1.02		0.99	1.06	1.06	1.03	1.06
Dry density, lb per cu ft	61.0		68.8	84.0	61.5	71.4	94.4
Moisture content as tested, percent	1.7	7.3	3.1	5.0	2.5	3.5	4.5
Thermal conductivity[b], k	1.87	2.31	2.29	3.01	1.77	2.27	3.42
Computed value of thermal transmission U[a], for 8-in. wall	0.20	0.24	0.24	0.29	0.19	0.23	0.32

[a] U is the number of BTU per hour transmitted through each sq ft of specimen for each degree F difference in temperature between the air on the two sides for a condition of still air on the warm side and a 15-mph wind on the cold side.

[b] k is the number of BTU per hour transmitted through each sq ft of specimen for each degree F temperature difference in the material per inch of thickness.

were used, one being placed on each side of the hot plate and between it and the adjacent cold plate. The heat flowing through a central metering area 4 in. square was measured and the heat conductivity of the material computed. The average mean temperature of the hot-plate specimens was approximately 68 F. Immediately after testing, these specimens were dried for 24 hours at 230 F in order to ascertain the moisture content of the concrete at the time of test.

Test results—The results of the observations on heat transfer tests are given in Table 6. The heat transmission is expressed in terms of the over-all coefficient of heat transmission, U, of the 8-in. wall, and thermal conductivity, k. Both heat transfer coefficients were computed for each type of test to provide a comparison of values obtained by the two methods. It is believed that the moisture content of the large walls was considerably greater than that of the small specimens used in the hot plate tests and that the higher values obtained in the hot box tests are due to this greater moisture content. The magnitude of this effect is indicated in Table 6 by the two hot plate tests made at different moisture contents on the burned-shale specimen leanest in cement.

Thermal expansion coefficients

Six specimens, 1½ in. square and 9 in. long, were cut from each kind of concrete. Data were obtained with a 6-in. Tuckerman optical strain gage operating on specimens placed in a temperature-controlled box equipped with observation portholes. Readings were made at two extremes of temperature, 104 F (40 C) and 14 F (−10 C), a full cycle requiring 24 hours. Measurements were made for a number of cycles until the readings indicated no progressive changes in length. The

TABLE 7—LENGTH CHANGES DUE TO TEMPERATURE, WETTING AND DRYING, AND DRYING SHRINKAGE

Concrete mix No.	1	2	3	4	5	6
Coefficient of thermal expansion, 10^{-6} degrees F	3.6	4.1	4.5	5.1	5.5	6.2
Shrinkage of 8-in. thick wallettes in laboratory air:						
At 1 year 10^{-6} in. per in.	−440	−180	+160[a]	−800	−710	−360
At 2 years, 10^{-6} in. per in.	−660	−410	+160[a]	−960	−1060	−710
Changes in length[b] during wetting and drying at 70 F, 10^{-6} in. per in.	−270	−500	−600	−400	−630	−970

[a]The plus sign denotes an expansion of 0.000160 in. per in.
[b]Values signify shrinkage on drying cycle and are equal and opposite to expansion on wetting.

temperature of the air surrounding the specimens was determined with calibrated copper-wire resistance thermometers and was controlled with a bi-metal thermoregulator within limits of ±0.2 F.

The coefficients of thermal expansion were determined for six like specimens and the average for each concrete is reported in Table 7.

SHRINKAGE CHARACTERISTICS

Shrinkage was measured on wallettes 8 x 30 x 30 in. Vertical changes beginning at an age of one week were obtained with a 20-in. Whittemore strain gage. The specimens were stored in laboratory air and measurements were made over a period of two years.

Values for change in length due to wetting and drying were also obtained. These measurements were made using a vertical comparator on specimens 1½ in. square and 7 in. long; the specimens were cut from larger pieces of the concrete. When 18 to 24 months old, they were fully wetted for a week in water at 70 F, then dried to constant length at 70 F and 40 percent relative humidity. They were then immersed for 48 hours and again dried to constant length, several of these cycles being made. Measurements of changes in length were noted from the start, and it was found that there was but a slight change from cycle to cycle.

Values of shrinkage in laboratory air and those of length change during wetting and drying are shown in Table 7.

RAIN PENETRATION

Test specimens

The six specimens used for the water permeability tests, one of each mix, were about 51 in. high, 41 in. wide, and 8 in. thick. They were built on supporting channels with a 2-in. mortar base containing a copper flashing projecting at the back. The surfaces were not finished

in any way; the walls were tested at an age of 3 months in their original condition.

Test equipment and procedure

The water-permeability test is described in BMS82,* "Water Permeability of Walls Built of Masonry Units." The specimens were supported on metal skids, and when clamped in position, the exposed face formed one side of a pressure chamber. An air pressure of 10 lb per sq ft above atmospheric pressure was maintained in the chamber, and water from a perforated tube was sprayed near the top edge of the exposed face at the rate of 40 gal. per hour.

Test results

The arbitrary method of rating the performance as given in BMS82 was employed. All the walls were judged to be excellent because no visible water appeared above the flashings in 1 day, and in 5 days there was no leakage and less than 25 percent of damp wall area.

SUMMARY

The leanest mixtures (No. 1 and 4) made with each aggregate had compressive strengths of 500 to 800 psi and dry weights of 60 to 65 lb per cu ft. Walls of this low-cement, high-void concrete possess heat transfer properties suitable for insulation in some types of buildings. Hot plate test values of k equal to 1.8 and 1.9 BTU compare very favorably with thermal conductivity values of 10 to 16 for ordinary dense concrete.

Heat transfer, compressive strength, and density all increased with increase in cement content, but thermal conductivity k increased only to 3.0 and 3.4 for the dense mixtures, which is still a relatively low value for monolithic concrete.

Resistance of walls to rain penetration was shown to be excellent.

Tests show that concrete with a compressive strength of 6000 psi can be made by using burned shale aggregate; the expanded-slag mixture with a cement content of 11 sacks per cu yd had a compressive strength of 3500 psi. The ratio of compressive to transverse strengths was about the same as for ordinary sand and gravel concrete although more cement was required with the lightweight aggregates to obtain equivalent strengths. The modulus of elasticity was 2,170,000 psi for a burned shale aggregate concrete of 5000 psi compressive strength.

Bond strength between the concrete and the steel reinforcement was about the same as for ordinary dense aggregate concrete of the same compressive strength.

*Available from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., for 20 cents.

Coefficients of thermal expansion for the lightweight aggregate concretes made with burned shale ranged from 3.6 to 4.5 $\times$ 10^{-6} per degree F, and from 5.1 to 6.2 $\times$ 10^{-6} for those made with expanded slag. All of the mixtures showed normal shrinkage ranging from 0.02 to 0.10 percent except the burned shale concrete richest in cement. This exhibited an expansion of almost 0.02 percent when stored in laboratory air for a period of one year. Length changes during wetting and drying ranged from 270 to 600 $\times$ 10^{-6} in. per in. for the burned shale aggregate concretes and from 400 to 970 $\times$ 10^{-6} in. per in. for the expanded slag aggregate concretes; they were smallest for the lean concretes and largest for the rich concretes.

ACKNOWLEDGMENT

The heat transfer properties were determined by H. E. Robinson of the National Bureau of Standards heat transfer section; the water permeability properties were measured by C. C. Fishburn of the masonry construction section.

Letters from Readers takes over from the former **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present ideas (the most valuable things we have) for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

HARLAN H. EDWARDS rebels against ACI authors who "hide" trade names when reporting test data in their papers. DOUGLAS E. PARSONS agrees in part, but presents the case for concealment.

R. H. SHERLOCK answers a question from Australia—how to fasten SR-4 gages on outside surfaces of concrete.

STANTON WALKER suggests mixing techniques which will eliminate occurrence of cement balls in concrete.

FRANK RICHART differentiates between f_a and f_b as used in the Joint Committee Report—resolving the dilemma of a University of Colorado engineering student.

Why Conceal Trade Names? (LR 45-6)

asks HARLAN H. EDWARDS, consulting engineer, Seattle, Washington, in the following plea for more explicit information.

This is a plea for completeness of reporting in papers and discussions published in the JOURNAL, instead of the time-worn, useless concealment of the trade names of materials and equipment by non-informative letters or numbers. Time after time reports are made or articles are written containing information which would be of value to the reader if only he knew what product the author was talking about. To refer to a material under test as "Admixture A" or another as "Type 1, a proprietary brand" or still another as "Type 2 is a commercial product" without giving their commonly known trade names is in reality an insult and an exasperation to the reader.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 2, Oct. 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

Why is it that this practice has arisen particularly in the technical press? If it is the fear of giving the materials a little free advertising of their use and performance, the time devoted to testing and writing about it should warrant that courtesy. If it is lack of space in tables, then the identifying information should be given in an appendix. If it is the fear that an unfavorable report on a material should not be given openly, then the writer and the JOURNAL are as guilty of concealing information that the reader should know for his own protection as the manufacturer may be for marketing an inferior product under a misleading label or for otherwise misrepresenting his product.

The American Concrete Institute and the pages of its JOURNAL should be the source of unbiased information about materials, methods and equipment used, just as *Consumers Research Bulletin* and similar magazines are or try to be to their subscribers. The engineer, architect, or the man on the job is not blessed with readily available, unlimited information and must depend upon the JOURNAL and other similar publications for fair, truthful and complete reporting. Where else can they get it in these days of untruths and half-truths in advertising? No, the Federal Trade Commission cannot be of much help because of insufficient appropriations for the purpose. Could lobbyists have persuaded Congress to reduce FTC appropriations?

From the reader's standpoint, time devoted to reading and analyzing a report is valuable and is compensated for by the information gained. He has little or no time, usually, to write the author and ask for the data. If he does, by the time it arrives, if ever, the trend of thought is probably gone and its maximum value is lost. In Mr. Wuerpel's very valuable article on "Concrete Containing Air Entraining Admixtures" (ACI JOURNAL, February 1946, p. 305) Table 1 gives the nature of the admixtures and the report details their effect in concrete. Several admixtures were evidently trade products. Admixture V was developed and patented by Mr. Wuerpel—evidently one of the best of the group—yet there was no information telling when and where it could be obtained. All of the report was intensely interesting and valuable, but what good was it to other than a laboratory man or a manufacturer who knew the characteristics of his competitor's material? The man on the job was stopped by the concealment of the very information he needed.

Often, the characteristics of a material described in a test report will suggest to the reader another application or use for which it might be fitted. Even though the material or equipment may not have proved best for the work under discussion, if the trade name is told, preferably also with the name of the manufacturer, an inquiry can be readily made that may prove that the product is of the utmost value in some other way. That has happened to me.

It is refreshing and satisfying, therefore, to find the article by J. W. Kelly and Bernerd D. Keatts on "Two Special Methods of Restoring and Strengthening Masonry Structures" (ACI JOURNAL February, 1946) telling the entire story of the job—what was used, when, why and how. Lets have more of them.

DOUGLAS E. PARSONS, chairman of ACI's Technical Activities Committee, replies to Mr. Edwards.

EDITOR'S NOTE—Mr. Parsons' letter was written in reply to Mr. Edwards' plea and not in enunciation of a complete policy of TAC for or against the publication of brand names. TAC has adopted no such policy. He might be said to express the views of authors and their principals, that harm may be done in some cases by a reader's unwarranted confidence that a brand name necessarily carries assurance of unchanging properties in a material. Mr. Parsons writes to ACI: "I doubt if anyone would object to the inclusion of brand names in ACI papers, unless by so doing a paper would become of value to those who wish to make unwarranted claims in advertising."—Thus the subject is wide open for further discussion.

Members of the Technical Activities Committee agree that authors often do not tell the readers enough about the materials and equipment which they have investigated. If brand names, together with other easily recognizable designations, were sufficient identification for nearly all commercial products, we think that authors would be delinquent if they failed to cite the brand names. But experience has taught us that physical properties, composition and performance characteristics of commercial products are changed frequently and without the knowledge of many purchasers thereof even though they continue to bear the same brand names. Many examples could be cited. Each time that a general survey has been made of the properties of waterproofings, admixtures for concrete, or plastic calking materials there were a number of such changes made after the samples were obtained and before the reports were published. And long before a report on a later survey was published, the original samples of most of the materials were not representative of the product being sold under the same brand name.

A manufacturer should not be condemned for changing his product either when forced to do so because of inability to obtain one or more of the raw materials or when the change is intended to improve the quality of the product. Even if all such changes were called promptly to the attention of the purchaser, brand names often would not afford reliable guides for the purchaser because of a lack of uniformity between different samples of products of the same brand. Until the producers of such products regularly check the qualities of the raw materials used and apply control methods that assure reasonable uniformity of their products, the publishing of brand names is more apt to be misleading than helpful information to purchasers.

Fastening SR-4 Gages on Outside Surfaces of Concrete (LR 45-7)

is the problem faced by DAVID V. ISAACS, director, Department of Works and Housing, Chatswood, N.S.W. (Australia).

In reading "Protection of Electric Strain Gages in Concrete," (Nov. 1947 JOURNAL) by R. H. Sherlock and Adil Belgin, we notice that reference is made to "a considerable amount of experience in the use of gages on the outside surfaces of concrete."

We are very interested in this application of gages but anticipate trouble from moisture in the concrete causing a leakage resistance through the adhesive used for cementing the gages on to the surface, and difficulty in securing a smooth enough surface for intimate contact with the gage.

R. H. SHERLOCK, professor of civil engineering, University of Michigan, Ann Arbor, draws on personal experience to answer the query.

These gages have previously been used at the University of Michigan on the outside surfaces of concrete on a model of a pavement slab, on a floor slab, and on a project conducted by my collaborator, Adil Belgin. The first of these projects has a progress report published as follows: "A Model Study of Slab Action in Concrete Pavements" by L. D. Childs, physical research engineer, Michigan State Highway Department, *Proceedings* Highway Research Board, 1945, pp. 45-48.* This publication does not contain much detail of the technique of using the gages. The investigator, however, refers to the article "Strain Gages" by D. M. Nielsen, *Electronics*, December 1943, as his source of information regarding the characteristics of these gages.

The second of these projects is reported only in thesis form by M. H. Makzoumi, now in Beirut, Lebanon. The report on the third project is now in the form of a doctoral dissertation by Mr. Belgin, and will probably appear in published form within the coming year.

One of our graduate students is now trying to develop methods of embedding these gauges in the plastic concrete without attaching them to the reinforcing steel and protecting them with the plastic shields as described in our article. However, he has only begun this project, and there is nothing to report now.

A general procedure for applying the SR-4 gages to concrete surfaces is as follows:

1. Provide a clean and smooth surface for the gage by using sandpaper or a grinding stone or both. Extreme smoothness is not necessary since later steps insure a proper bond between the gage and the concrete

*Available from the Division of Engineering and Industrial Research, National Research Council, 2101 Constitution Ave., Washington, D. C.

surface at all points. Too much grinding may loosen some of the pebbles near the surface and make it impossible for the gage to give an accurate indication of the deformations of the undisturbed concrete.

2. Apply a liberal amount of Duco cement to the surface to be occupied by the gage.

3. Immediately embed the gage in the surface of the Duco cement and apply a uniform pressure for two or three minutes. About 2 lb per gage is sufficient.

4. Allow the cement to set for at least 24 hours before attaching the electric wires leading to the indicator. Here again all the usual precautions should be observed to obtain reliable contacts and to prevent grounding or short circuits.

Cause and Cure for Cement Balls in Concrete (LR 45-8)

R. D. RADER, San Francisco sends the following question:

What is the cause and what is the remedy for lumps or balls of cement or cement and sand appearing in concrete from ready mix plants and from stationary mixers when the concrete appears otherwise to be well mixed?

One answer comes from STANTON WALKER of Washington, D. C.

The cause of lumps or balls of cement in mixed concrete is not readily explained in terms to satisfy all of the conditions under which they occur. In a general way they are tied in with the manner in which mixing water comes in contact with the cement. If the cement is poorly distributed when the water reaches it, the relatively slow mixing action may permit the absorption of enough water by portions of the cement to form lumps tenacious enough to resist breaking up during mixing, especially for wet consistencies.

For the general case, two procedures, which seem to add up to about the same thing, appear to have been successful in preventing the formation of these lumps or balls. One is to mix the concrete thoroughly at a slump of less than about 3 in. (the lumps not so readily forming at consistencies this dry) and, then, if a wetter consistency is required, add such additional mixing water as is needed, accompanied by adequate additional mixing of not less than about 20 revolutions of the drum. The second, which would seem to be substantially the same as the first in effect, is to add mixing water to the batch slowly with the drum revolving.

The likelihood of lumps forming may be increased by various circumstances. Cement with a tendency to be water repellent may aggravate

the condition. Hot cement is frequently blamed, and no doubt sometimes is the cause. In at least one case, where the difficulty was probably contributed to by hot cement, the formation of the lumps ceased when the mixing water was cooled.

What Is the Difference Between f_a and f_b? (LR 45-9)

questions CHRISTIAN W. MATTHEWS, student at the University of Colorado, Boulder.

This is a request for information regarding the definition and calculation of the stress, f_b, as used in Eq. (18), Section 870, p. 88, of the 1940 Joint Committee Report.

A considerable difference of opinion exists here on the campus in our concrete structures class. Some have recommended using f_a of Eq. (17) for the stress f_b. I can offer no alternative suggestion, but it seems unreasonable to make this substitution for by definition:

f_a = *average* allowable compressive stress
f_b = allowable working stress.

I hope that you will find it possible to clear the confusion which exists regarding this matter.

FRANK E. RICHART, University of Illinois professor and former member of the Joint Committee, explains:

I am sure that it was intended that f_a as defined on p. 84 of the report and f_b as defined on p. 88 mean the same thing, the average allowable compressive stress in the concrete portion of the column. It is true that a column might be designed to have a stress less than the allowable value f_a, and in this case Eq. (18) becomes somewhat conservative, but we assumed that the designer would generally use as near the allowable stress as he possibly could.

Probably the two different symbols were used for the same stress because the chapter on columns and the one on footings were written by different subcommittees, and the difference in notation was overlooked.

Eq. (18) is generally used to determine the size of a pier or pedestal used between the column and the footing slab. I understand that such pedestals are not used nearly as much as they were a few years ago, because of the tendency to simplify design to meet high construction costs. The equation is rather far-fetched (in more ways than one) since it was derived from tests by Bauschinger in Germany in 1876 (see Bach, "Elastizität und Festigkeit"). These tests were made on cubes of stone, loaded over only a portion of their area, corresponding to the part of a pedestal area covered by a column. However, the equation has been used in footing design for many years. It was given in the second Joint Committee Report, issued in 1924.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Design and reinforcement of circular concrete foundation slabs (Berechnung und Bewehrung von Kreisplatten [Stahlbeton] als Grundungsplatten)

WILHELM NERLICH, *Bauplan. u. Bautech.*, V. 2, Jan. 1948, pp. 5-9 — APPLIED MECHANICS REVIEWS April 1948, (Chenea)

Formulas for the bending and twisting moments and shear forces in a circular plate loaded symmetrically and unsymmetrically are collected for use in designing reinforced concrete foundation slabs and footings. Design curves are presented and layout of the reinforcing steel is discussed.

Vibrating of concrete during the setting process (Wibrowanie betonu w czasie jego wiazania)

W. OLSZAK, *Inzyn. Budown.*, March-April 1947, V. 4, pp. 177-182 — APPLIED MECHANICS REVIEWS June 1948, (Bernhard)

Compression and tension of concrete as a function of the time and procedure of vibration during setting have been experimentally obtained. A pronounced strength improvement, generally increasing with the length of the vibration process, has been noted.

The compound girder as combination of steel girder with reinforced concrete slab (Der Verbundtrager als Kombination von Stahltrager mit Eisenbetonplatte)

A. ALBRECHT, *Ingenieur, 's Grav.*, V. 59, Nov. 7, 1947, pp. 81-88 — APPLIED MECHANICS REVIEWS Feb. 1948, (Hetenyi)

The paper reviews work done, chiefly in Switzerland in the past 15 years, on compound beam constructions, in which steel girders and reinforced concrete slabs are joined together to form an integral load-carrying structure. The resulting construction combines the stiffness and low maintenance cost of concrete structures with the easy erection, lighter weight and better reinforcing possiblities of steel structures.

The principal problem in the design of such compound girders is the transfer of longitudinal shear from girder to slab, which is best accomplished by angles welded to the steel girders and serving as dowels. The paper reports the results of a series of tests of such dowels on specially prepared shear specimens, and of static and fatigue tests on complete small-span compound-girder constructions. Numerical examples of calculation are given, with applications to building and bridge construction, including a bibliography on the subject.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 2, Oct. 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

Prefabs on Parade

112 pp., The Housing Institute Inc., 527 Fifth Avenue, New York 17, N. Y., 1948, $2.00

Following a very brief introduction this 8⅝ x 11-in. book is illustrated on nearly every page with views of the manufacture of prefabricated units and photographs and illustrations of almost every type of prefabricated home. Floor plans and typical interiors are also shown. Each house picture is accompanied by notes on unusual construction details, fixtures included, and materials supplied by the manufacturer.

Graphical determination of bending moments in continuous beams

A. J. Ashdown, *Concrete and Constructional Engineering*, (London) V. 43, No. 6, June 1948, pp. 161-170 Reviewed by Glenn Murphy

A simple graphical method, based on the theorem of three moments, for determining the bending moments in beams is described by the author.

The method outlined may be used for concentrated, uniformly distributed, or triangular loadings. According to the author it gives results to an accuracy compatible with the original assumptions involved in the theorem of three moments.

Highway Research Board Proceedings 1947

Roy W. Crum, Fred Burggraf and W. N. Carey, Jr., editors, Highway Research Board, Washington, D. C.

This 523 page book records the proceedings of the 27th annual meeting of the Highway Research Board held at Washington, D. C., December 2–5, 1947. A total of about 50 papers, reports and discussions are published on a range of subjects including: economics, finance and administration; design; materials and construction; maintenance; traffic and operations; and soils investigations. Also listed are papers and reports presented at the meeting but published elsewhere.

Composite construction for I-beam bridges

C. P. Siess, *Proceedings* American Society of Civil Engineers, V. 74, March 1948, pp. 331-353
Applied Mechanics Reviews April 1948, (Volterra)

The results of analytical and experimental studies of composite construction for I-beam highway bridges which are presented in the paper may be divided into three groups: (1) comparative design studies of I-beam bridges, to determine the savings in weight possible from the use of various types of composite beams; (2) studies of the behavior of composite beams; (3) studies of the behavior of shear connectors for use in composite I-beam bridges.

Shell concrete construction

H. G. Cousins, *Technical Paper* No. 6, The Reinforced Concrete Association (London) June 1948, 32 pp.
Reviewed by C. P. Siess

This paper contains a general discussion of shell concrete roof construction. Domes are mentioned briefly but the emphasis is on barrel vaults of the Zeiss-Dywidag type. Various types of construction are described. Design is covered briefly and equations are given for the stresses in the shell. Some tests of shell roofs are mentioned. Details of construction are discussed at length including: expansion joints, waterproofing, insulation, lighting, ventilation, concrete, reinforcement, formwork, costs, and prestressing of the edge beams. There are several pages of discussion.

Contribution to the calculation of arch bridges in reinforced concrete with rigid arch and deck (Bijdrage tot het berekenen van boogbruggen van gewapend beton met stijve boog en stijf rijdek)

J. F. A. Broeckhoven, *Ingenieur,'s Grav.*, V. 59, No. 40, Oct. 3, 1947, pp. 73-79
Applied Mechanics Reviews
Jan. 1948, (Boiten)

An approximate calculation of the above-mentioned type of bridge is presented under the assumptions: (1) that the number of suspension bars is infinite; (2) that the elongation of the suspension bars, of the webbing, and of the arch itself is zero. The assumptions are discussed, corrections are given, and the errors are estimated.

Creep and shrinkage in reinforced concrete structures

H. B. Seed, *Reinforced Concrete Review* (London), V. 1, No. 8, Jan. 1948, pp. 253-267
Reviewed by C. P. Siess

The nature of creep and shrinkage is explained on the basis of "the solid solution or colloidal theory of cement composition," and the seepage of water through the gel as a result of pressure gradients due either to moisture conditions or to applied loads. Four procedures for computing movements due to creep are presented: (1) the reduced effective modulus method, (2) Glanville's rate of creep method, (3) a superposition method suggested by W. H. Lorman, and (4) a method developed by A. D. Ross using a mechanical model incorporating both elastic and viscous elements. Creep movements calculated by the various methods are compared with each other and with the results of tests.

Thermal stresses in arch dams (Le sollecitazioni indotte dalle variazioni de temperatura nells dighe ad arco)

Filippo Arredi, *Giornale del Genio Civile* (Rome) V. 85, Feb. 1947, pp. 55-70
Applied Mechanics Reviews
April 1948, (Maulbetsch)

In this article expressions for thermal stresses in thick-walled circular cylinders are derived and used for the case where the temperature of one surface is a sinusoidal function of the time. Considering dams as sections of cylinders the author finds values for thermal stresses, which he compares to empirical data used in Italy for dam computations. Conditions are assumed to be uniform along the height of the dam. The only edge effect considered is that of the supports on the sides.

Concrete sleepers in Belgium

Concrete and Constructional Engineering, V. 43, No. 6, June 1948, pp. 179-181
Reviewed by Glenn Murphy

Two types of sleepers or ties are described. One, the Blaton sleeper, consists of two end blocks 2 ft 6½ in. long and 7½ in. deep connected by a section approximately 9½ in. wide and 3 in. deep. The three parts are cast monolithically and are reinforced by a ½-in. bar bent and welded to form a link about 6 in. wide and 7 ft long. The link is stretched in the mold before the tie is cast. After the concrete has hardened the stretching force is released, inducing longitudinal compression in the tie. The concrete is placed by vibration. Several thousand ties of this type have been in service on the main lines of Belgian railways for over two years.

The second type of tie also consists of three blocks. These are cast separately and assembled by a longitudinal steel bar passing through the units and stretched to induce initial compression in the tie. This type of tie has also been in use on the main railway lines in Belgium for nearly two years and is said to combine the characteristics of high stability under static loads and high elastic deformation in service.

Reinforced concrete flats at Walthamstow

Concrete and Constructional Engineering, V. 43, No. 6, June 1948, pp. 182-184

Reviewed by GLENN MURPHY

A series of housing units constructed at Walthamstow in Essex is described. Units consist of 20 six-story blocks and one two-story block occupying an area of about 21 acres and providing housing for 414 family units.

A typical six-story block is about 130 ft long and 29 ft wide with reinforced concrete construction used throughout. Temperature effects are provided for in the design, but the structures contain no expansion joints.

Wood wall-lining is provided for insulation and soundproofing. Construction details are described.

Development of the strength of concrete

W. E. ABBOTT, *Concrete and Constructional Engineering*, V. 43, No. 6, June 1948, pp. 177

Reviewed by GLENN MURPHY

The author shows that the strength-age relationship for concrete may be expressed in the form

$$S = C_1 (1 - e^{-k_1 t}) + C_2 (1 - e^{-k_2 t})$$

in which S is the compressive strength

t is the age

C_1 and C_2, k_1 and k_2 are constants.

Values of the constants are determined for a series of mixtures recently reported by the Road Research Laboratory and very close agreement is indicated.

Vibration of fresh concrete (La vibration de beton fraiz)

R. L'HERMITE AND G. TOURNON, *Annales* de L'Institut Technique du Batiment et des Trauvaux Publics, No. 11 New Series, Feb. 1948

Reviewed by C. P. SIESS

This circular presents the results of extensive theoretical and analytical research on the effect of vibration on the properties of fresh concrete. The theoretical studies by L'Hermite attempt to relate the various characteristics of the vibration, such as energy of vibration and rate of propagation, with fundamental physical properties of the concrete, such as the coefficient of internal friction and viscosity. The experimental investigations by Tournon included studies of the effect of vibration on the workability of concrete as measured by slump and flow tests as well as by other methods, ring shear tests to determine the coefficient of internal friction of vibrated concrete, measurements of the viscosity of concrete in a state of vibration and studies of the segregability of concrete.

Experimental proportioning of concrete mixtures (Determination experimentale de la composition du beton)

M. BUISSON, *Annales* de L'Institut Technique du Batiment et des Travaux Publics No. 21 New Series, May 1948

Reviewed by C. P. SIESS

The author describes a procedure for proportioning concrete to obtain a desired strength with a given set of aggregates. The method is fundamentally similar to the mortar-voids method of Talbot and Richart. The relation between the cement-space ratio and compressive strength is used, and the aggregates and paste are proportioned on the basis of absolute volumes. A device patterned after the Proctor soil compaction equipment is used to determine the water content corresponding to the greatest density of the sand alone and also to determine the density of a sand, gravel and water mixture. The proportion of sand to gravel to give the desired density is determined by trial.

Laboratory research on concrete bridge floors

F. E. RICHART, *Proceedings* American Society of Civil Engineers, V. 74, March 1948, pp. 288-304
APPLIED MECHANICS REVIEWS
April 1948, (Volterra)

A program of systematic research was begun in September 1936 with the purpose of making investigations and tests of reinforced concrete slabs to determine their behavior under varying conditions, and to develop information which will advance the art of bridge building.

A combination of analytical and experimental study has been employed throughout the work with the main object of covering three definite phases: (1) mathematical analysis along conventional lines; (2) experimental research on structures, with results analyzed and compared with theoretical findings; (3) the development of simplified design procedures for rapid and easy use.

New investigations of the freezing of concretes: air-entraining concrete (Nouvelles recherches sur la gelivite des betons: les betons aeres)

M. VALENTA, *Annales* de L'Institut Technique du Batiment et des Travaux Publics, No. 22 New Series, May 1948 Reviewed by C. P. SIESS

This paper discusses analytically and with reference to fundamental physical properties the problems associated with freezing of concrete. Consideration is given to such factors as temperature gradient through non-homogenous materials, humidity, thermal expansion, thermal conductivity, time rate of temperature variation, porosity, permeability, and absorptivity. Improvement of resistance to freezing is discussed from the point of view of the cement, aggregates, proportions, and various wetting, dispersing, and air-entraining agents. Results are given from tests to determine the hydrostatic pressures generated when concrete is frozen and also from tests of the strength of frozen concrete. Two appendices contain extensive reviews of the literature (mostly American) on freezing of concrete and on the properties of air-entraining concrete.

New method for investigating the relaxation of steel wires (Une nouvelle methode pour l'etude de la relaxation des fils d'acier)

M. G. DAWANCE, *Annales* de L'Institut Technique du Batiment et des Travaux Publics, No. 9 New Series, Feb. 1948 Reviewed by C. P. SIESS

A new method of studying the relaxation properties of small diameter steel wires used for reinforcement in prestressed concrete is described. The wire is clamped between fixed points of a frame with a known initial stress, and the load at any subsequent time is determined by measurement of the fundamental transverse frequency of vibration. Typical results are given for tests up to 800 days on cold-drawn hard steel wires having diameters of 0.10 and 0.20 in., and ultimate strengths of 284,000 and 228,000 psi, respectively. It was concluded that an initial stress of two-thirds the ultimate would be reduced approximately 10 percent by relaxation in 10 years. A brief theoretical treatment of relaxation is presented. In conclusion, the author discusses the differences between the conditions of the test and those in a prestressed concrete member.

Does concrete need expansion joints?

W. S. FOSTER, *Civil Engineer Corps Bulletin* V. 2, No. 20, July 1948

The author discusses briefly past and present engineering practice and theories which dictate the design of contraction and expansion joints in concrete pavements. Based on the discussion he draws the following conclusions: (1) The term "contraction joint" is a misnomer. These joints are the result of pavement warping from temperature and moisture differences in the pavement and should normally occur at 15- to 20-ft

intervals in unreinforced concrete. (2) Under normal conditions, paving does not require expansion joints. Pavement is as long as it is going to be when poured and still plastic, unless poured in abnormally cold weather. (3) If the design uses weakened plane joints at relatively short intervals, no load-transfer dowel bars are required. The irregularities of the fracture will be sufficient to transfer the load. (4) If the pavement end is unsupported and does not abut against existing pavement, there will be a tendency for the short sections to work their way apart, sufficiently so that the aggregate interlock will not transfer the load. To correct this the writer suggests including load-transfer dowel bars in the hard joints for the last 500 ft of the project. (5) A similar situation exists when the pavement approaches a bridge abutment. Ample expansion should be allowed so that the slab cannot introduce a horizontal stress on the abutment, and to supply dowels for 500 ft from that point.

A bibliography of 6 items is listed.

Studies in cement-aggregate reaction

Bulletin No. 229, Council for Scientific and Industrial Research, Commonwealth of Australia (Melbourne) 1947

Reviewed by DELMAR L. BLOEM

Part I—Australian aggregates and cements

A. R. ALDERMAN, A. J. GASKIN, R. H. JONES and H. E. VIVIAN

Observations of mortar bars containing a large number of aggregates and cements have been correlated with results of petrographic examination of the aggregates to determine which combinations of aggregate and cement may react detrimentally in concrete. While in most cases potential reactivity can be determined from petrographic analysis alone, some borderline aggregates may require supplementary tests of mortar bars for their assessment.

Part II—Effect of alkali movement in hardened mortar

H. E. VIVIAN

It is demonstrated by means of composite mortar bars that alkalies in mortar tend to migrate from areas of high alkali concentration to areas of low concentration. Alkalies from high alkali cement were found to move through as much as ½ in. of mortar (made with innocuous materials) to react with a reactive aggregate in another section of the specimen. The importance of this phenomenon in regard to controlling alkali aggregate is discussed.

Part III—Effect of void space on mortar expansion

H. E. VIVIAN

Increasing the void space (air content) of mortar made with reactive combinations of cement and aggregate was found to reduce the expansion of mortar bars. Void space was varied by changing the proportions of cement, water and aggregate—not by air entrainment in the usual sense. It was found that mortars having a void content in excess of 7 percent showed no appreciable expansion even though evidence of reaction between cement and aggregate was discernible on broken sections of the specimens.

Part IV—Effect of expansion on the tensile strength of mortar

H. E. VIVIAN

The author found that tiny cracks produced by alkali-aggregate reaction greatly reduced the tensile strength of mortar before abnormal expansions occurred. He discusses the use of tensile strength tests as a possible means of identifying reactive combinations of materials more quickly than can be done by expansion measurements of mortar bars. He also uses the relationship between tensile strength and expansion as a clue to the mechanism of cracking caused by alkali-aggregate reaction.

Part V—Effect of void space on the tensile strength changes of mortar
H. E. VIVIAN

Pursuing further the subjects discussed in Parts III and IV of this bulletin the author found that increasing the void space in mortar decreased the reduction in tensile strength caused by alklai-aggregate reaction.

Part VI—Effect of carbon dioxide
A. J. GASKIN

Disruptive expansion of mortar bars made with reactive materials was virtually eliminated by storing the bars in an atmosphere of water-saturated carbon dioxide. A minimum storage period of 28 days in 90 per cent pure carbon dioxide was required to insure the prevention of expansion at later ages. Treatment with carbon dioxide was unsuccessful in preventing reduced tensile strengths of mortars made with reactive materials. Attempts to use sulfur dioxide in place of carbon dioxide failed because of the rapid deterioration of mortar in the presence of this gas.

Possible cuts in highway costs—A symposium

Engineering News Record, V. 141, No. 2, July 8, 1948, pp. 120-127 Reviewed by S. J. CHAMBERLIN

More mechanization is best bet for cutting construction costs
DWIGHT W. WINKLEMAN

More cooperation between equipment manufacturers in the development of improved machinery and methods; more cooperation between distributors and contractors in the maintenance of equipment; and more cooperation between engineers and contractors in the design and supervision of projects are necessary to realize the maximum potentials of new machinery. Contractors can work with equipment manufacturers on the design and development of improved equipment. Systematic maintenance and prompt repair are needed with increased mechanization, and the recently established Joint Cooperative Committee between the Associated Equipment Distributors and the Associated General Contractors of America is proving helpful in this work. The Joint Cooperative Committee of the American Association of State Highway Officials and the A.G.C. has been working in the field of design to permit increased mechanization. Unnecessary items must be eliminated from plans and specifications to cut costs.

Three cost cutting tips from Texas
D. C. GREER

Costs may be cut by : (1) preparing designs that permit forms to be used over and over again for both substructures and superstructures; (2) maximum utilization of local materials; (3) elimination of refinements in design that are not absolutely necessary.

Cutting costs in the mid-west
GEORGE C. KOSS

Cooperation among the various agencies has made it possible to revise specifications, eliminate restrictions on size of equipment, change methods of curing, eliminate rubbing of unexposed concrete and reduce hand finishing. Two 34-E dual-drum pavers per crew with other highly mechanized equipment makes for mass production. The large investment in construction equipment, however, necessitates larger projects and a long working season.

Fewer restrictions on initiative will reduce road costs
MORRIS E. DEWITT

Competition fosters efficiency, but some agencies cannot get sufficient bids because of unsatisfactory experiences that the contractors have had with them. Larger projects are needed to maintain competition among substantial contractors. Standardization of structures and of the entire design of a highway will aid in attaining greater efficiency

and will effect economies in materials and supplies. The contractor should have full opportunity to apply his ingenuity in devising and developing new and better methods. Anticipation of needs for repairs and general items and renewed accident prevention work will lower costs for the contractor.

Savings in cost result from encouraging specialization

R. C. KEELING

Bridge plans in Kansas have been standardized as much as possible to simplify formwork and permit its reuse. Concrete has been used more extensively because of the shortage of steel. On grading projects hauls have been kept short, ditches have been designed for heavy equipment operation, and less finishing is required. Local materials have been used more extensively and larger projects have been let. Bids have been taken separately on bridges, grading and surfacing to allow for specialization and to encourage small contractors.

Specifying results, not methods, will cut construction costs

LEET M. DENTON

Sensible, understandable construction specifications with adequate plans promote competition. Efficient, well maintained equipment, together with the best personnel will lower costs. The contractor should be left free to develop new methods without being hampered by restrictive methods of construction. Design engineers and specification writers should become more familiar with actual field operations. Rights-of-way should be acquired promptly. Pipe culverts should be considered as substitutes for box culverts and box culvert designs should be more standardized. Standardization of wire mesh would result in economy.

Lack of real competion results in higher bids

J. S. BRIGHT

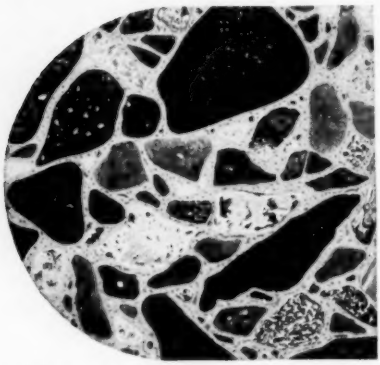
Closer cooperation between the equipment manufacturer, the contractor and the highway engineer will speed up equipment modernization that will reduce the cost of highway construction. Lack of bids indicates that the contracting industry is operating near capacity. Much work is behind schedule because contractors are being offered more work than they can perform efficiently and they have taken on more jobs than they can handle properly. Some contractors do not figure their bids closely and allow too much for contingencies. Knowledge that competition will not be keen, removes all risk from bidding and without an element of risk, competitive bidding is meaningless. It may be expedient, if contract prices continue to rise, to resort to force account operations for some types of work. The Public Roads Administration has urged state highway departments to establish these three primary prerequisites as a basis for making contract awards: (1) the improvement must be urgent in character; (2) there must be adequate competition in bidding; and (3) the unit prices must be justified by proper engineering analyses.

ACI News Letter

Vol. 20 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE October 1948

PREPAKT REPORTS . . .

. . . ON STRUCTURAL STRENGTH IN CONCRETE



PREPAKT Concrete shows an internal sectional pattern of tightly compacted aggregate bound into a monolithic structure.

ORDINARY concrete shows an internal pattern of suspended and dispersed aggregate with frequent water-filled voids under aggregate.

The rock-like structure of PREPAKT Concrete, producing exceptional bonding and high compressive strength, plus superior resistance to cracking and deterioration, is obtained as follows:

1. Carefully graded and thoroughly cleaned coarse aggregate is placed in such manner that every cubic yard of PREPAKT Concrete contains 27 cubic feet of coarse aggregate after compaction.
2. This highly compacted mass is pressure grouted with specially prepared and patented binding materials of high penetrability to transform the aggregate into a monolithic structure.

These two factors produce extremely dense concrete, composed of more strength-imparting material than ordinary concrete. Here is a field comparison of typical PREPAKT Concrete and typical ordinary concrete prepared for similar use:

MATERIAL IN 1 CUBIC YARD

Material	Unit	Ordinary Concrete	Prepakt Concrete
Coarse Aggregate	Cubic Feet	18—21	27 (Compacted)
Fine Aggregate	Cubic Feet	12—14	6
Portland Cement	Sacks	6	4
Alfesil (Mineral Filler)	Pounds	0	150
Conditioning Aid	Pounds	0	5
Water	Gallons	32—35	28—30

HIGH RESISTANCE TO FREEZING-THAWING • LESS CRACKING
LOWER PERMEABILITY • RESISTANCE TO SULPHATE MOISTURE
HIGH COMPRESSIVE STRENGTH • HIGH BONDING STRENGTH

THE CONCRETE WITH PREPAKT EXTREME DURABILITY

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ACI

News Letter Contents

October 1948

On the cover—A photograph of the sculptor's model of the Alfred E. Lindau Award plaque. The cover picture is approximately the same size as the actual award. Story on p. 5.

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Nominating Committee reports: Gilkey, president Thomson, vice president

Directors: H. E. Davis, Delzell, Newmark and R. W. Spencer

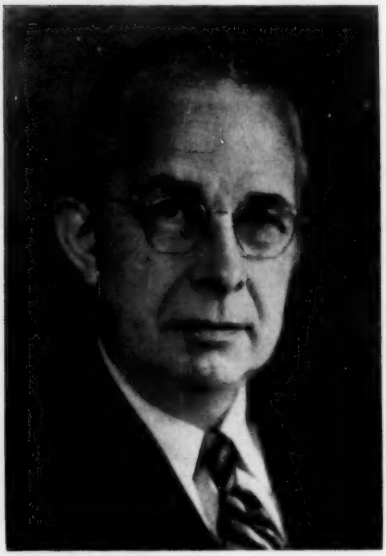
The 1948 ACI nominating committee, consisting of Roy W. Carlson, chairman, Raymond E. Davis, Harrison F. Gonnerman, Douglas E. Parsons, Frank E. Richart, Thomas E. Stanton, Stanton Walker and Charles S. Whitney has reported the following nominations:

HERBERT J. GILKEY for president to succeed Robert F. Blanks for a one-year term beginning at the February 1949 convention.

Professor Gilkey is head, Department of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa; member of the Institute since 1924; Wason Medalist for the most meritorious paper, 1939; member of the Standards Committee since 1942 and chairman since 1944; member Publications Committee 1939-1946; member of the board of direction as director, former sixth district, 1937-38 and elected to the same position 1945-46; elected vice president 1947; re-elected vice president 1948.

HARRY F. THOMSON for vice president, 2-year term, beginning at the February 1949 convention.

Mr. Thomson has been an ACI member for 20 years; member of its Publications Committee 1940-41, of the board of direction as director 1939-40; director-at-large since 1942. He entered the concrete field in 1927 as vice president of the newly organized General Materials Co. of St. Louis, and as a ready-mixed concrete manufacturer joined ACI next year. Following the death of A. C. Butterworth late



Presidential nominee
Herbert J. Gilkey

in 1947 he was elected president of the General Materials Co.

For directors, 3-year terms, four nominations for as many places on the Board: HARMER E. DAVIS, H. C. DELZELL, NATHAN M. NEWMARK and R. W. SPENCER.

Mr. Davis is associate professor of civil engineering, University of California, Berkeley, member of ACI since 1930, Wason Medalist for the most meritorious paper of 1931; chairman of ACI Committee 620, Construction Joint Practice and active in the work of several other committees.

Mr. Delzell, as managing director of the Concrete Reinforcing Steel Institute, Chicago, has represented that organization in ACI for the last 8 years. C.R.S.I.'s membership dates from 1927.

Professor Newmark, of the College of Engineering, University of Illinois, has been an Institute member since 1934; was co-author with R. E. Copeland of "Structural and Economic Studies of Monolithic Concrete Walls for Dwellings," ACI *Proceedings* V. 31; a member of the Technical Activities Committee; a member of the executive group of ACI Committee 115, Research; chairman of ACI Committee 321, Design of Reinforced Concrete Slabs.

Mr. Spencer, chief civil engineer of the Southern California Edison Co., Los Angeles, has been an ACI member since 1932; is author of "Measurement of the Moisture Content of Concrete," ACI *Proceedings* V. 34, and chairman of Committee 604, whose report on "Recommended Practice for Winter Concreting Methods" was recently adopted as an Institute standard (ACI 604-48); is active on three other ACI committees.

Vice presidential candidate
Harry F. Thomson

Twenty candidates for membership on the 1948 nominating committee—5 to be elected—are:

R. E. COPELAND	W. H. KLEIN
R. W. CRUM	G. L. LINDSAY
R. E. DAVIS	H. S. MEISSNER
J. R. DWYER	F. R. McMILLAN
ALEX. FOSTER, Jr.	F. W. PANHORST
W. C. HANNA	GERALD PICKETT
FRED HUBBARD	F. E. RICHART
HERBERT INSLEY	B. W. STEELE
J. W. KELLY	I. L. TYLER
H. L. KENNEDY	M. O. WITHEY

ACI Announces Alfred E. Lindau Memorial Award

The Alfred E. Lindau Award of the American Concrete Institute for recognition of noteworthy contributions to the development of reinforced concrete design was founded in 1947 by the Concrete Reinforcing Steel Institute subject to conditions of the founder as accepted by the American Concrete Institute as follows:

The award shall be given only for outstanding contributions to reinforced concrete design practice and is not mandatory each year. Any and all persons, firms or corporations are eligible to compete for and receive the award. The funds to establish and maintain it in perpetuity shall be limited to the funds subscribed by the Concrete Reinforcing Steel Institute. Except as provided in the foregoing stipulations, the procedure for administration of the award shall be in accordance with the American Concrete Institute's established practice governing awards.

Token of award

The token of the award is a bronze plaque bearing a bas-relief portrait of Alfred E. Lindau, past president, 1924-25, of the American Concrete Institute who died at Pearl Harbor December 14, 1944 after five years of service as chief civilian engineering assistant to the Officer in Charge, Pacific Naval Air Bases. In that position he gave a range of engineering service and assumed responsibilities exceeding the implications of his official title, in assisting and advising the officers of the Civil Engineer Corps of the navy from his arrival at Pearl Harbor December 1, 1939 to the day of his death.

The plaque (see *News Letter* cover) provides an area in which awardee names, dates and specific citations will be engraved.

Origin of award

The establishment of an award in Lindau's memory had its inception with a small group from among his many personal friends in ACI. They planned to finance it by many individual subscriptions. The first check to start the fund came to ACI from Franklin R. McMillan who, as secretary of the third Joint Committee on Concrete and Reinforced Concrete, had worked long and closely with Lindau, the committee's chairman. (Because of subsequent developments the sum subcribed was returned to him.) H. C. Delzell, managing director, Concrete Reinforcing Steel Institute, for which Lindau in the thirties had served as consulting engineer, came into the memorial award conversations. He presented the matter to the board of directors of the C.R.S.I. which undertook the entire financing of the award. From them ACI received $3,800.00 as an award fund to be administered by the Institute first for the initial art work on a mutually approved design and a principal sum whose revenue is to perpetuate the award.

NEW YORK CONVENTION PLANS

ACI 1949 New York convention plans are maturing—likely to reach approximate crystallization at meetings of the ACI Technical Activities Committee and Board of Direction in October. Numerous program items are promised or prominently in sight but hardly at announcement stage—two of the featured items now in hand in completed manuscript, four others due in October, all items scheduled for availability by November 1. An almost complete announcement of New York subject matter and personnel may be looked for in the November JOURNAL.

Pavement problems

B. D. Tallamy, chief engineer, division of construction, State of New York Department of Public Works, and Charles M. Noble, chief engineer, New Jersey State Highway Department, will present the regional concrete paving problems of construction and maintenance as a basis for general discussion. Both these contributors are highly competent to wrestle with a situation which is of the widest possible interest not only to road builders but to students of concrete looking to solution of problems of durability under severe exposures.

Air entrainment

The present state of knowledge of air entrainment will go before the convention in a brief summarization by J. F. Barbee, Ohio State Highway Department, of this much-written-about and discussed subject. Late developments of research, job experience and practice will form still another paper in early prospect.

Architectural concrete

"Cast-in-place architectural concrete" will be presented in two papers on design elements and construction practice—by John J. Hogan on design and E. B. Oberly on construction, each collaborating in his separate paper with A. J. Boase. Authors and co-authors are all of the Portland Cement Association.

Slab design

C. P. Siess and N. M. Newmark will make brief presentation of a new basis of reinforced concrete slab design. (See "Rational Analysis and Design of Two-Way Concrete Slabs," scheduled for pre-convention publication in the December JOURNAL as advance basis of discussion having an important bearing on the development of future design practice.) The structural session will be built up further from three or four items now being considered by TAC.

Lightweight concrete

Lightweight aggregate concrete will be discussed in a report to be made of National Bureau of Standards studies.

Committee 115—Research

The Research Committee will have its usual session; selection of topics will be based on returns from Committee 115's annual questionnaire to many research agencies in the field of concrete.

Other items are in prospect, final program selections to be made on the basis of probable convention interest, brevity of presentation and availability early enough to make possible general announcements for advance build-up of intelligent discussion.

The Technical Activities Committeee and the Board of Direction will meet in separate sessions on Tuesday, February 22.

Technical committee meetings

ACI Technical committees have been asked to make early report of their intentions with reference to meetings in the forenoon February 23. Accommodations have been blocked out to take care of all likely comers in the way of committee reservations.

The general sessions of the convention will begin at two o'clock Wednesday, February 23. The convention will end with the adjournment of the session Friday afternoon, February 25.

New York committee

Right now the New York Convention Committee, under the chairmanship of M. J. McMillan, is getting set for its job. It is a job of many details in letting the engineering public of the northeast and contributing areas know what to expect of the convention in the way of information on top topics. The job also includes details of New York arrangements which only a local group can handle effectively. That committee consists of: E. K. Abberley, Waldo G. Bowman, J. H. Chubb, A. B. Cohen, Roger Corbetta, Joseph Di Stasio, Homer Farmer, Elliott A. Haller, Frank L. Kelly, Emil Praeger, Myron A. Swayze, B. H. Wait and C. A. Willson.

Who's Who This Month

Frank E. Richart

An ACI member since 1917, past president of the Institute and Wason Medalist in 1938, Frank Richart scarcely needs editorial introduction in these pages This time he appears as author of "Reinforced Concrete Wall and Column Footings" the first part of which appears on p. 97. His impressive record of long-time service to the Institute includes work as committeeman, administrator, and author of some 16 papers and reports. He has been active in A.S.T.M. and A.S.C.E. work and has written numerous bulletins for the University of Illinois Engineering Experiment Station. He is a member of the Society for Experimental Stress Analysis, Western Society of Engineers, Tau Beta Pi, Sigma Xi and Phi Kappa Phi.

Douglas McHenry

Technical Activities Committee member Douglas McHenry was introduced to News Letter readers in June as the author (with W. T. Walker) of "Laboratory Measurements of Stress Distribution in Reinforcing Steel." His new JOURNAL contribution (p. 129), "Lattice Analogy in Concrete Design" gives the application of a simplified mathematical solution of two dimensional stress problems.

Educated at the University of Minnesota, with a background of public utilities work and dam design, Mr. McHenry became head of the structural research section at the Bureau of Reclamation Laboratories, Denver, in 1943. He is a member of Sigma Xi, A.S.C.E., American Institute of Physics and has been affiliated with ACI since 1936.

Jacob Feld

New York consulting engineer Jacob Feld draws from his professional experience to report tests of "Strength of Precast Concrete Floor Joists," p. 141 of the October JOURNAL. A native of Austria, Dr. Feld received his engineering education in the United States at the College of the City of New York and the University of Cincinnati, and has been active in consulting work since 1925. The Sixth Avenue Subway in New York, Naval Ammunition Depot at Earle, N. J. and the Stewart Airfield at West Point, N. Y. are among his more important consulting jobs in the past decade.

A member of Phi Beta Kappa, Sigma Xi and Sigma Tau Phi, Dr. Feld has written many scientific and technical papers for German, Australian and American periodicals, including "Precast Concrete Pit Sheeting" in the ACI JOURNAL, April 1945. He is a member of A.S.C.E., a fellow of the American Association for the Advancement of Science, member Brooklyn Engineers Club, New York Academy of Sciences, and an associate of the Highway Research Board. A registered professional engineer in New York, he became a member of the American Concrete Institute in 1944.

Paul Klieger

"Effect of Entrained Air on Concretes Made with So-Called 'Sand-Gravel' Aggregates" p. 149, is contributed by Paul Klieger, ACI member since 1942. Mr. Klieger graduated in civil engineering at the University of Wisconsin in 1939, and during the following 2 years was employed by the Soil Conservation Service of the U. S. Department of Agriculture. In 1941 he joined the Portland Cement Association staff as assistant engineer, entered the army in 1942, and in 1945 returned to his P.C.A. position.

Perry H. Petersen

National Bureau of Standards tests form the basis of Perry H. Petersen's report on "Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture," p. 165. In March this year Mr. Petersen, who joined ACI in 1946, became a research associate for the Structural Clay Products Institute in charge of their work at the National Bureau of Standards. He has been with the bureau's masonry construction section since 1937.

A native New Yorker, Mr. Petersen is a civil engineering graduate of the Cooper Union Institute of Technology. He has written several Building Materials and Structures reports published by the Bureau of Standards, but this is his first ACI contribution.

PROFESSIONAL CARDS

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Construction Methods, Supervision

W. G. Kaiser promoted

W. G. Kaiser, nationally known agricultural engineer and ACI member since 1935, has been appointed assistant director of promotion for the Portland Cement Association. In his new capacity Mr. Kaiser will work with another Institute member, W. D. M. Allan, director of promotion. An agricultural engineering graduate of Iowa State College and a member and past president of the American Society of Agricultural Engineers, Mr. Kaiser was awarded the A.S.A.E. Cyrus Hall McCormick medal in 1946 for meritorious engineering in agriculture. T. E. Bond succeeds Mr. Kaiser as manager of the P.C.A. farm bureau.

Nolan D. Mitchell to head fire protection section

Nolan D. Mitchell, Institute member since 1921, has been appointed chief of the Fire Protection Section of the National Bureau of Standards, where he will direct research for the building technology division on problems of fire resistance in relation to the design and construction of buildings and building material. A civil engineering graduate of the University of Arkansas in 1908 and an architectural engineering graduate of the University of Illinois 1910, Mr. Mitchell has been on the staff of the National Bureau of Standards since 1922. Outstanding among his important contributions in the fire protection field is his investigation of the control of airplane hangar fires by the automatic application of water. His research on the fire resistance properties of structural materials including metal frames and wood frames, interior partitions and columns protected by gypsum, has lead to the determination of the effect of fire and the time in which a wall, floor or other structural element will retard the spread of fire. In addition to his ACI affiliation, Mr. Mitchell is a member of the Washington Society of Engineers, A.S.T.M., the National Fire Protection Association and Sigma Xi.

New Members

Of 58 applications for membership received in August and approved by the Board of Direction 34 were Individual, 5 Corporation, 8 Junior, 11 Student.

The geographical listing shows Texas in front for the month with 7 new applicants. California and Costa Rica each provided 6 new applicants.

Our membership total September 1, 1948 became 4261.

Arkansas

Shivley, James M. (Indiv.) P. O. Box 63, Mountain, Home, Ark.

California

Campomanes, Napoleon V., (Jr.) c/o Robert & Co., Inc., APO 246, c/o P. M. San Francisco, Calif.

Hartley, Alexander S., (Indiv.) 224 Nuevo St., Fontana, Calif.

Jackman, Thelma C., (Indiv.) Los Angeles Public Library, Serials Division, 630 W. 5th St., Los Angeles 13, Calif.

McClure, Frank E., (Jr.) 2530 College Ave., Berkeley 4, Calif.

Nummy, George M. (Indiv.) c/o Paraffine Companies, Inc., P. O. Box 709, Redwood City, Calif.

San Bernardino County Flood Control Dist. (Corp.) 115 E. Sixth St., San Bernardino, Calif. (Roy L. Taunton)

Colorado

Rao, U. Ananda, (Indiv.) Olin Hotel, Denver, Colo.

Connecticut

Mozzochi, John Joseph (Indiv.) 379 Nawbuc Ave., Glastonbury, Conn.

Florida

Borowski, James N. (Indiv.) Box 868, Palm Beach, Fla.

Illinois

Kashef, A. I. (Indiv.) 201 Engineering Hall, University of Illinois, Urbana, Ill.

Poston, William E. (Indiv.) Poston Brick & Concrete Products Co., 2600 E. South Grand Ave., Springfield, Ill.

Stallmeyer, James E. (St.) 419 Newman Hall, 604 E. Armory Ave., Champaign, Ill.

Kansas

Hillabrant, James G. (St.) 823 N. 8th St., Manhattan, Kans.

Massachusetts

Lovell, John M. (Indiv.) 26 May St., East Braintree, Mass.

Michigan

Dunkirk, George (Indiv.) 7249 Kentucky, Dearborn, Mich.

Nebraska

Fahy, F. M. (Indiv.) U. S. Engineer Office, Republican City, Nebr.

New Jersey

Van Breemen, William (Indiv.) RFD No. 1—Upper Ferry Road, Trenton, N. J.

New York

Chapman, Edward A. (Indiv.) Rensselaer Polytechnic Institute Library, Troy, N. Y.

Rabinowitz, W. (Indiv.) Western Union Telegraph Co., 60 Hudson St., Rm. 1806, New York, N. Y.

Sih, Nan-sze (Jr.) 1135 John Jay Hall, Columbia University, New York 27, N. Y.

North Dakota

Larson, John W. (Indiv.) 807 8th St., Bismarck, N. D.

Ohio

Herbster, D. H. (Indiv.) c/o The Neff & Fry Co., 150 S. Main St., Camden, Ohio

Hilkert, W. A. (Indiv.) 100 N. High St., Akron, Ohio

Lausell, Demetrio (Jr.) 84 W. 2nd Ave., Columbus 1, Ohio

Lethly, Marlay W. (Indiv.) 31 New Zimmerman Bldg., Springfield, Ohio

Nyquist, Roy A. (Indiv.) Engineering Dept., Technical Div., Libby-Owens-Ford Glass Co., 1701 E. Broadway, Toledo 5, Ohio

Oregon

Steinbrugge, Henry S. (Jr.) 7114 S. E. 17 Ave., Portland 2, Ore.

Pennsylvania

Kindig, Carl H. (Indiv.) Bucknell University, Lewisburg, Pa.

South Dakota

Agrimson, Donald M. (St.) Pickstown, S. D.

Texas

Badgett, Joe S. (Jr.) 255 Oak Grove Courts, Austin 22, Texas

Correa-Reyes, Jose J. (Jr.) Texas Highway Dept., Houston, Texas

David, A. S. (Indiv.) 6323 Allegheny, Houston, Texas

Fischer, Stewart C. (St.) 1336 Lamonte Lane, Houston, Texas

LaFoy, E. L. (Indiv.) 4611 Cole Ave., Dallas, Texas

Mallett, Edward Franklin (Indiv.) 2131 Jean, Houstin 3, Texas

Reid, Robert L. (Indiv.) 1705 W. Gray St., Houston 6, Texas

Virginia

Schafer, H. J. (Indiv.) Dept. of Engineering, The Engineer School, Fort Belvoir, Va.

Washington

Noyes, Charles W. (Indiv.) c/o Pre-Mix Concrete Inc., 110 N. Washington St., Kennewick, Wash.

Wyoming

Erzen, Ben T. (Indiv.) 1312 Ninth St., Rock Springs, Wyo.

Puerto Rico

Ramos, Santiago Rodriguez (Indiv.) Caohillas Hydroelectric Project, Utuado, Puerto Rico

Argentina

Weibel, Arthur (Indiv.) Dewey & Almy (Argentina) SAIC, Primera Junta 550, Quilmes FCS Argentina

Belgium

Societe Belge Des Betons (Corp.) 3F Boulevard Du Regent, Bruxelles, Belgium (Raymond Mercier)

Brazil

Fragoso, Paulo Rodrigues (Indiv.) Avenida Erasmo Braga 277, 13°-s. 1301, Rio de Janeiro, Brazil

Canada

Neufeld, James C. (Indiv.) 809 Spruce St., Winnipeg, Manitoba, Canada

MacInnis, Cameron (Jr.) c/o Hydro-Electric Power Comm. of Ontario, 8 Strachan Ave., Toronto, Ont., Canada

Costa Rica

Alvarado, Alejandro (St.) San Vicente de Moravia, Costa Rica, C. A.

Cordero L., Carlos (St.) Casa No. 43F., Av. 14, San Jose, Costa Rica

Padilla, Manuel Antonio (St.) Ciudade la del Zapote, Casa No. 4, Pabellon No. 6, San Jose, Costa Rica, C. A.

Rivera, Edison (St.) Box 950, San Jose, Costa Rica, C. A.

Roman Jara, Carlos Maria (St.) San Isidro de Coronado, Republica, Costa Rica

Truque, Fernando Gonzalez (St.) Box No. 1365, San Jose, Costa Rica, C. A.

England

Roxana Ltd. (Corp.) Queen's Mill Road, Huddersfield, Yorkshire, England (Tom Pickles)

Pakistan

Huq, Mohammad Rafiqul (Indiv.) c/o Chief Engineer, Dept. of Works & Bldgs., Govt. of East Bengal, Dacca, Pakistan

Shafie, Mohammed, (Indiv.) Palm, Pedro D'Souza Road, Karachi 5, Pakistan

Portugal

Laboratorio De Engenharia Civil, (Corp.) Avenida Rovisco Pais, 41, Lisbon, Portugal (Eduardo De Arantes E Oliveira)

South Africa

Glauber, Jan (Jr.) 2 Mindale Court, Minors St., Yeoville, Johannesburg, S. Africa

Messrs. John Laing & Son (S. Africa) Pty. Ltd. (Corp.) Maritime House, Loveday, St., Johannesburg, S. Africa (H. K. Sladkus)

Now available on request to those who ask for it is an 8-page pamphlet reprinting the synopses of all ACI papers and reports published in Journal V. 19 — Sept. 1947 to June 1948

Honor Roll

February 1 to August 31, 1948

Prof. Newlin D. Morgan, Jr., University of Wyoming, retains the lead in our current Honor Roll for the period ending August 31 with credit for 30 new members. Number two on the list is Prof. Newlin D. Morgan, University of Illinois, with just half as many member credits as his son, but with Prof. Raymond E. Davis and Thomas E. Stanton,

Morgan, Sr.

Morgan, Jr.

both close at his heels to make clear California's go-getting ways.

On our Honor Roll this month are 270 member names representing the 6 percent of ACI's total membership (4261) who have been "member conscious" this year. Will some of the other 94 percent be persuaded to enlist in the effort?

Newlin D. Morgan, Jr. (Wyo.).....30
Newlin D. Morgan (Ill.).....15
Raymond E. Davis (Calif.).....14½
Thomas E. Stanton (Calif.).....14
Harrison F. Gonnerman (Ill.).....5
Henry L. Kennedy (Mass.).....5
James A. McCarthy (Ind.).....5
Howard Simpson (Mass.).....5
Wm. H. Thoman (Colo.).....4½
Raphael Callejas H. (Guatemala).....4
E. I. Fiesenheiser (Ill.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Leon Venegas (Costa Rica).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
Robert W. Freeman (Calif.).....3
A. E. Mackney (Canada).....3
F. N. Menefee (Mich.).....3
Robert Morris (Colo.).....3
Richard A. Roberts (Calif.).....3
G. E. Troxell (Calif.).....3
Joseph J. Waddell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
John C. Sprague (Ga.).....2½
Hugh Barnes (Calif.).....2
F. Campus (Belgium).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
Ray C. Giddings (Calif.).....2
Ernst Gruenwald (N. Y.).....2
Myle J. Holley, Jr. (Mass.).....2
Edward L. Howard, Jr. (Calif.).....2
R. R. Kaufman (Ohio).....2
John C. King (Ohio).....2
George E. Large (Ohio).....2
William E. Lumb (Canada).....2
Rene S. Pulido y Morales (Cuba).....2
Hugh E. Odor (Wash.).....2
James M. Polatty (Ga.).....2
Walter H. Price (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
John A. Ruhling (Ill.).....2
C. H. Scholer (Kans.).....2
E. Copeland Snelgrove (England).....2
H. D. Sullivan (Wash.).....2
J. Neil Thompson (Texas).....2
E. W. Thorson (Colo.).....2
Stanton Walker (D. C.).....2
Jose Luis Capacete (Puerto Rico).....1½
Milton Fromer (Calif.).....1½
Thomas C. Kavanagh (Pa.).....1½
Eugene Mirabelli (Mass.).....1½
W. E. Parker (Canada).....1½
Dean Peabody, Jr. (Mass.).....1½
Jerome M. Raphael (Colo.).....1½
Clarence Rawhouser (Colo.).....1½
F. E. Richart (Ill.).....1½
Edward W. Scripture, Jr. (Ohio).....1½
Roy T. Sessums (La.).....1½
Harry F. Thomson (Mo.).....1½
John Tucker, Jr. (D. C.).....1½
Calvin T. Watts (La.).....1½
Harold E. Wessman (Wash.).....1½

Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
A. Amirikian.....1
Boyd G. Anderson.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1
F. Barona.....1
Robert F. Blanks.....1
A. J. Boase.....1
E. K. Borchard.....1
H. L. Bowman.....1
H. E. Burr.....1
John Butkus.....1
A. Carbone.....1
Harold S. Carter.....1
A. N. Dutta Choudhury.....1
C. O. Christy.....1
Miles N. Clair.....1
Rodger B. Collons.....1
R. Torres Colondres.....1
Jack E. Counts.....1
Walter Dardel.....1
J. Slater Davidson, Jr.....1
Harmer E. Davis.....1
George C. Ernst.....1
A. Finlayson.....1
T. H. Fleming.....1
R. E. Fowle.....1
Frank M. Fucik.....1
Carmelo Galindez.....1
Anthony J. Giardina.....1
H. J. Gilkey.....1
R. R. Gore.....1
Emil A. Gramstorff.....1
Eugene Guillard.....1
Samuel Hobbs.....1
W. C. Huntington.....1
Phil C. Huntly.....1
Wm. R. Johnson.....1
H. D. Jolley.....1
Edward F. Keniston.....1
P. J. Kennedy, Jr.....1
Orla A. Larsen.....1
B. Leon.....1
W. G. Limbach.....1
L. D. Long.....1
Charles Macklin.....1
John V. Maescher.....1
Clyde P. Mason.....1
F. T. Mavis.....1
William G. McFarland.....1
Douglas McHenry.....1
F. R. McMillan.....1
Carl A. Menzel.....1
L. J. Meszaros.....1
Oliver H. Millikan.....1
Leonard J. Mitchell.....1
Robert B. B. Moorman.....1
John J. Mullen.....1
R. P. Mumford.....1
T. D. Mylrea.....1
Amado B. Navoa.....1
William T. Neelands.....1
C H. Neill.....1
N. M. Newmark.....1
D. G. Norman.....1
Wm. D. Nowlin.....1
George L. Otterson.....1
R. A. Plumb.....1
James J. Pollard.....1
Horace A. Pratt.....1
Abdul Qayum.....1
R. D. Rader.....1
Thomas J. Reading.....1
J. W. Rivett.....1
H. H. Roberts.....1
Fernando A. Rojas B.....1
Georges Ronai.....1
H. C. Ross.....1
E. Gonzalez Rubio.....1
C. E. Sandstedt.....1
Edgar A. Sealey.....1
C. Seshavatharam.....1
Thomas C. Shedd.....1
Joseph Sill.....1
Francis P. Sloan.....1
Gustave Spirz.....1
Hale Sutherland.....1
E. O. Sweetser.....1
T. Thorvaldson.....1
Dean P. Tsagaris.....1
Juan Agustin Valle.....1
Walter C. Voss.....1
Jukka E. Vuroinen.....1
W. F. Way.....1
Paul Weidlinger.....1
J. M. Wells.....1
C. D. Williams.....1
Merle D Wilson.....1

The following credits are, in each instance "50-50" with another member.

Robert W. Abbett
George C. Alden
Oscar Vega Arguelles
A. Allan Bates
E. W. Bauman
C. T. Bishop
Petrus Manzano Bonilla
P. G. Bowie
Harold W. Brewer
A. R. Brickler
Boyd S. Brooks
L. Rees Brooks
Sterling Lowe Bugg
B. W. Burns
Jose Luis Casas
Thomas F. Chace
Wilbur H. Chamberlin
Richard de Charms
R. V. Chate
J. E. Christiansen
James H. H. Chun
H. W. Chutter
Emil W. Colli
A. R. Collins
F. Coppen
Rolland Cravens
J. M. Crom
R. A. Crysler
Charles W. De Groff
John G. Dempsey
J. B. Dolsen
J. E. Dunn
J. Dvorak
John N. Eckle
Harlan H. Edwards
A. C. Eichenlaub
F. L. Ehasz
Arthur D. Engle
William E. Feidt
Rudolph Fischl
E. M. Fleming
Leo V. Garrity
Carl V. Gezelius
Luis Alberto Truque Gurdian
J. R. G. Hanlon
Bernard Harrison
Frank W. Haws
H. L. Henson
A. E. Hiscox
W. C. Hodgkins
John Hogan
G. W. Holcomb
H. N. Howe
Walter L. Huber
J. W. Hubler
Manuel Castro Huerta
Orville E. Jack
Frank H. Jackson

M. E. James
Bruce Jameyson
E. B. Johansen
Harry D. Jumper
Thomas M. Kelly
John J. Kennedy Jr.
K. J. Krawczyk
H. M. Larmour
Robert E. Lee
L. G. Lenhardt
John F. Long
Wallace A. Marsh
F. R. McCabe
Hubert F. McDonell
Hugh Montgomery
Ray M. Moorhead
Israel Narrow
Elmer H. Oechsle
Francisco Santos Oliva
Chester B. Palmer
G. S. Paxson
Donald O. Peck
Ben Poisner
Chesley J. Posey
Emil H. Praeger
Herman G. Protze
Warren Raeder
Dwight F. Roberts
Bernard L. Robinson
Fredric Rusche
Fred H. Ryan
Anton Rydland
Fernando Sanchez
Herman Schorer
Joseph J. Shideler
Charles A. Shirk
Chester P. Siess
R. Simons
J. Singleton-Green
Charles M. Spofford
A. R. Stuckey
M. O. Sylliaasen
Ruth D. Terzaghi
A. G. Timms
Donald R. Warren
J. C. Watt
J. D. Whittaker
William H. Wisniski
Harry C. Witter
Roderick B. Young

Three new publications for home builders have been released

Warm, dry concrete floors

How to build a warm, dry concrete floor for a basementless house is one of the problems receiving intensive research study at the University of Illinois during the past year. Nine different types of floor slabs were tested in a special laboratory in the project carried out by the Small Homes Council and the Mechanical Engineering Department at the university. Recommendations forthcoming for dry floors are: (1) good drainage with a 4-in. fill of coarse washed gravel or crushed rock under the floor slab; (2) a vapor barrier consisting of a ⅛-in. rigid asphalt board or of reinforced duplex paper with asphalt center, over the fill and extending to the outside edge of the floor; (3) at least 2 in. of rigid waterproof insulation along the exposed edge of the floor and extending 2 ft under it. The complete findings are reported in a 4-page nontechnical circular entitled "Concrete Floors for Basementless Houses," which is available without cost for a limited time. Requests should be addressed to Small Homes Council, Mumford House, University of Illinois, Urbana.

Basement problem weighed

"Basement vs. No Basement" is the title of a series of articles appearing in the Housing and Home Finance Agency *Bulletin* issued periodically in Washington, D. C. The paper on basements appearing in No. 3, 4 and 5 of the bulletin includes data on cost reduction through functional requirements, modular coordination and condensation remedies. These bulletins are available upon request from the H.H.F.A.

Engineering principles applied to home design

Home design by engineering principles will make possible the use of non-conventional building materials that provide sufficient strength with minimum material and labor, according to research reported in a new book entitled "Strength of Houses: Application of Engineering Principles to Structural Design" by Herbert L. Whittemore, John B. Cotter, Ambrose H. Stang, and Vincent E. Phelan. With the new design procedure intensities of service loads are first estimated, and then each material is selected to serve a specific function and to provide adequate strength at minimum cost. In the research program reported, compressive, transverse and racking loads were computed for typical homes in several locations representative of extreme wind and snow loads in the United States. Allowable safe loads for 100 wall, partition, floor and roof constructions were then compared with actual assumed loads. Comparison showed that some had insufficient strength while others were much stronger than necessary.

In addition to convenient computational methods for load distribution the book includes fundamental data for the designer on the wind, snow and occupancy loads that are likely to be imposed on a home structure. Issued by the National Bureau of Standards, the large size 132-page book is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. at $1.50 per copy.

Technical Committee Projects and Personnel

An important new ACI committee in the highway field and changes in the chairmanship and in the personnel of other committees prompt announcements here presented to make it convenient for you to correct your "ACI Directory 1948" in the Technical Committees section, pp. 196-213.

Heading the list is the recently organized Committee 325, Structural Design of Concrete Pavements for Highways and Airports under the chairmanship of L. W. Teller, who has called its first meeting in Washington, at the National Academy of Sciences, December 3, 1948.

Committee 611, Inspection of Concrete, still headed by Prof. J. W. Kelly as author-chairman, has added six new members to the five members announced in the 1948 Directory, p. 207. Chairman Kelly has already gone to his members with a first call to service in the revision, probably with some new material, of the ACI Manual of Concrete Inspection which has been a "best seller" since its publication in 140-page small pocket format in 1941 and still in large demand after several reprintings. Ahead of the committee is the objective of a new report suitable for adoption as an ACI standard. The original work meanwhile will continue on the Institute's list of publications.

Committee 325—Structural Design of Concrete Pavements for Highways and Airports

L. W. TELLER, Chairman
(Public Roads Administration, Washington, D. C.)

HENRY AARON*
(Civil Aeronautics Administration)

L. M. ARMS
(Portland Cement Association)

E. A. FINNEY
(Michigan State Highway Dept.)

BENGT F. FRIBERG
(Granite City Steel Co.)

A. T. GOLDBECK
(National Crushed Stone Association)

T. J. KAUER
(Wire Reinforcement Institute)

H. L. LEHMANN
(Louisiana Dept. of Highways)

P. E. MASHETER*
(Ohio Department of Highways)

NATHAN M. NEWMARK
(University of Illinois)

L. A. PALMER*
(Bureau of Yards and Docks, Navy Dept.)

G. S. PAXSON
(Oregon State Highway Dept.)

THOMAS B. PRINGLE*
(Office, Chief of Engineers, Dept. of the Army)

WILLIAM VAN BREEMEN
(New Jersey State Highway Dept.)

C. A. WILLSON
(American Iron and Steel Institute)

KENNETH B. WOODS
(Joint Highway Research Project, Purdue University)

F. N. WRAY*
(Highway Research Board)

The committee's preliminary assignment is to develop a recommended practice for the design of structural details for concrete pavement for highway and airport service—slab dimensions, cross-section, joints, reinforcement, etc., on the basis of the best information available,

*Non-Member ACI.

and to keep such recommendations up to date as new information indicates need for revision.

Committee 611—Inspection of Concrete

J. W. KELLY, Author-Chairman
(Engineering Materials Laboratory, University of California, Berkeley)

DONALD C. ANDREWS
(Turner Construction Co.)

MILES N. CLAIR
(Thompson & Lichtner Co., Inc.)

NOMER GRAY
(Walter Kidde Constructors)

FRANK H. JACKSON
(Public Roads Administration)

W. T. NEELANDS
(Corps of Engineers)

TREVAL C. POWERS
(Portland Cement Association)

WILFRED SCHNARR
(Hydro-Electric Power Comm. of Ontario)

R. W. SPENCER
(Southern California Edison Co.)

LEWIS H. TUTHILL
(Bureau of Reclamation)

I. L. TYLER
(Portland Cement Association)

RODERICK B. YOUNG
(Hydro-Electric Power Comm. of Ontario)

The committee's 140-page Manual of Concrete Inspection was distributed to the membership as of July 1941 and is available (for $1.00) to engineers, architects, contractors and others interested. This Manual, written with direct usefulness as the keynote, not only tells the concrete inspector what he should do and how to do it but also the underlying reasons for doing it that way. Within a few weeks the Manual went into first place as an ACI best seller, so that a second edition with limited revisions was issued in December 1941—since then three printings. In addition to a check list of the inspector's duties, the Manual contains a list of carefully selected reference material covering the unusual as well as the conventional phases of concrete construction, for more detailed study by the inspector when he gets into a tight spot. A comprehensive list of standard specifications and test methods rounds out the volume. More than five years in preparation, the Manual represents the combined experience and counsel of committee members representative of the varied Institute activities. Published as information and for discussion, it was given a place on the 1942 convention program. The resulting discussion (ACI JOURNAL, *Supplement*, Nov. 1942, p. 541) and subsequent reaction to and study of the original text are still the basis for an anticipated revised edition for consideration as an ACI standard. To speed the new work of the committee it has been reorganized with the personnel, new and old, as listed.

Two new committee chairmen are among other committee announcements. Raymond C. Reese heads Committee 315, Detailing Reinforced Concrete Structures, to fill the vacancy caused by the resignation of A. J. Boase who was responsible for bringing to public use information long needed. The Detailing Manual, recently advanced (unrevised) to the full status of ACI Standard, is in fresh and increased demand as an engineering text.

A. E. Cummings is the new chairman of Committee 323, Prestressed Reinforced Concrete—also as the successor of A. J. Boase.

Committee 315—Detailing Reinforced Concrete Structures

RAYMOND C. REESE, Chairman
(Consulting Engineer, Toledo, Ohio)

EARL P. ALLABACH
(Consulting Engineer, Philadelphia)

FRANK H. BEINHAUER
(Consulting Engineer, Davenport)

RENE L. BERTIN
(Consulting Engineer, Long Island City)

Continued on p. 19.

James Preston Hoskins

James Preston Hoskins, director, assistant secretary and assistant treasurer of General Portland Cement Co. died in Chattanooga, Tennessee, August 2, 1948 following a brief illness. Born Dec. 2, 1864 in Lexington, Miss., Mr. Hoskins attended preparatory school in Meridian, and Mississipppi Argicultural and Mechanical College in Sharksville. Following a banking career in Chattanooga he became associated with the Signal Mountain Portland Cement Co. as a director in 1923 and in 1933 he was made secretary and treasurer. In 1947 he became assistant secretary and assistant treasurer of the General Portland Cement Co.

Cement pricing decision

The Supreme Court decision June 7 to uphold the Federal Trade Commission ruling against delivered pricing in the Cement Institute case is discussed in an 8-page statement from the Chamber of Commerce of the United States. A legal analysis of the decision is presented with a brief reference to the program of the Capehart sub-committee of the Senate Interstate and Foreign Commerce Committee. Copies are available from the Chamber of Commerce of the United States of America, Washington 6, D. C.

R. F. Dierking advances

ACI member R. F. Dierking of Des Moines, Iowa was appointed manager of west central offices of the Portland Cement Association, effective September 15. Mr. Dierking, who has been district engineer in charge of the Des Moines office of the association since 1937 will have his new headquarters in Kansas City, Missouri. From that point he will direct the association's field work in Colorado, Kansas, Missouri, Nebraska, Oklahoma and Wyoming. An Institute member since 1937 and a former employee of the Missouri State Highway Commission, Mr. Dierking has been a member of the P.C.A. staff for more than 20 years.

American Standards Association incorporates

The American Standards Association became the American Standards Association, Inc. August 2 through incorporation under the laws of the State of New York. This change is third in a series consistently paralleling the enlarging scope of the association's work. The association was organized in 1918 as the American Engineering Standards Committee, a coordinating group for the standardization work of five of the country's important technical societies. Broadening of the organizational set-up to include government agencies led to an extension of the work in the field of safety standards so that in 1928 an entire reorganization took place changing the committee into the American Standards Association. The present change to an incorporated association gives it and its members the protection and benefits which corporation law affords and which are considered essential in light of the scope of the association's activities. Bills seeking federal incorporation of the association are now before Congress.

Highway board will meet December 7-10

The 28th annual meeting of the Highway Research Board will be held this year December 7-10 at the building of the National Academy of Sciences and National Research Council in Washington, D. C.

The Highway Research Board expects that this meeting will exceed in attendance the annual meeting of the board held in Washington last December when 853 persons registered, and 125 technical papers and reports were presented and discussed at 24 separate public sessions. Thirty-six state highway departments, 31 colleges, 26 government bureaus and 33 national associations were represented.

Positions and Projects — ACI Members

Dean Peabody, Jr.

Dean Peabody, Jr., ACI member since 1914, has joined the staff of the Division of Architecture of Harvard University, according to recent information from Cambridge. The appointment was effective with the 1947 academic year, but Professor Peabody, who had been teaching structural design at M.I.T., conducted classes at both institutions that year, making the complete transfer to Harvard effective this fall.

Co-author (with Howard R. Staley) of the Institute paper "Shrinkage and Plastic Flow of Prestressed Concrete," January 1946 JOURNAL, Professor Peabody is a member of ACI Committee 318. His other professional affiliations include membership in A.S.C.E., A.S.T.M., A.A. A.S. and the Boston Society of Civil Engineers.

John J. Manning

Rear Admiral John J. Manning, chief of the Navy Bureau of Yards and Docks, Washington, D. C., recently was elected a trustee of Rensselaer Polytechnic Institute and president of the Rensselaer Alumni Association. A Rensselaer graduate and member of the Civil Engineer Corps since 1918, Admiral Manning has held numerous important assignments, having been director of the Atlantic division of the bureau from November 1942 to June 1945, and director of the Eastern Pacific division from the latter date to January 1946.

Harry C. Plummer

Institute member Harry C. Plummer, director of engineering and research, Structural Clay Products Institute, was recently admitted to Keramos, national professional ceramic engineering fraternity, as an honorary member. The initiation was held at the North Carolina State College chapter, Raleigh. Mr. Plummer was selected in recognition of outstanding contributions to the brick and tile engineering profession in his present capacity with the Structural Clay Products Institute.

Herbert K. Cook

ACI member Herbert K. Cook has been named chief of the Concrete Research Division of the Waterways Experiment Station, Vicksburg, Mississippi.

Norman H. Withey

Norman H. Withey is now a member of the staff of the technical division, Newport Industries, Inc., New York, N. Y. He was previously associated with the Alpha Portland Cement Company, Easton, Pa. Mr. Withey was awarded the Wason medalist (with A. G. Timms) in 1935 for the paper "Temperature Effects on Compression Strengths of Concrete."

Delmar G. Runner

Delmar G. Runner has advised the Institute of his transfer from the Public Roads Administration, to the Non-Metals Division, U. S. Bureau of Mines, Department of the Interior, Washington, D. C.

Paul J. Halloran

Wason medalist (1944), Capt. Paul J. Halloran, who is retiring from the U. S. Navy Civil Engineer Corps, has been elected vice president of Foley Brothers, Inc., construction firm of Pleasantville, N. Y. During his 27 years of service with the navy, Captain Halloran has been in charge of several large construction projects for the Bureau of Yards and Docks. In World War II he commanded the assault Seabees in the Marianas campaign, and organized and headed the Sixth Naval Construction Brigade. His most recent assignment has been as district civil engineer for the Fifth Naval District.

Walter L. Couse

Walter L. Couse, contractor of Detroit, Michigan, has been appointed to the advisory council of the Senate Interstate Commerce Subcommittee on Trade Policies, which is investigating the effect on the national economy of Supreme Court and Federal Trade Commission price decisions. An ACI member, Mr. Couse, who is on the executive committee of the Associated General Contractors of America and chairman of the organization's market development committee, will present to the council the problems of the contract construction industry relating to pricing policies.

Minasian and Johnson

John K. Minasian and Carl B. Johnson have opened an office for the practice of civil and structural engineering in Los Angeles, California. Mr. Minasian joined ACI last year.

Harold S. Sweet

Harold S. Sweet, co-author with K. B. Woods of "Evaluation of Aggregate Performance in Pavement Concrete," June JOURNAL, and research engineer for the Indiana Joint Highway Research Project, received a doctor's degree in civil engineering from Purdue University at recent commencement exercises. With the exception of honorary degrees, this was the first doctorate in civil engineering awarded by the university.

Permanente officers re-elected

Henry J. Kaiser has been re-elected president and director of Permanente Cement Co., Oakland, Calif. Also re-elected were E. E. Trefethem, Jr., as vice-president and director; E. H. Heller, vice-president and director; Carl R. Olson, vice-president and general manager; G. G. Sherwood, treasurer and secretary; Paul E. Rogers, controller; L. S. Corey, H. A. Dick, H. W. Morrison and Gilbert J. Shea, directors. The Permanente company holds five contributing ACI memberships.

Leon D. Holden

Leon D. Holden has been appointed as project engineer by the Lehigh Portland Cement Co., Allentown, Penn. Formerly director of the Cement, Lime and Allied Plants Division of Vulcan Iron Works. Mr. Holden had served 18 years with the Lehigh company before leaving to do war work in 1942.

W. L. Shannon

W. L. Shannon, who became an Institute member in 1947, has moved from Waltham, Massachusetts to Portland, Oregon where he accepted a recently created position as acting director of the North Pacific Division Laboratory, Corps of Engineers, U. S. Army.

Charles R. Walker

Charles R. Walker, who received his master's degree in civil engineering at the Massachusetts Institute of Technology in June, has accepted the position of assistant professor of civil engineering at the University of Tennessee, Knoxville. He began his new duties at the opening of the academic year in September.

New Ceco offices

General offices of Ceco Steel Products Corp. manufacturer of metal construction products were moved this past summer to a new office building at 5601 West 26th Street, Chicago. The new general office building is of reinforced concrete construction with face brick to match the adjacent factory building. Concurrent with erection of the new office building Ceco also built a new factory addition to provide the company with a total of 360,000 square feet of manufacturing space in a building extending 1/5 of a mile along the main line of the Burlington Railroad.

ACI Nominating Committee report appears on p. 3 this month.

(*Continued from p. 15*)

EVERETT E. EBLING*
(Bethlehem Steel Co.)†

FRANK KEREKES
(Iowa State College)

T. D. MYLREA
(University of Delaware)

FRED L. PLUMMER
(Hammond Iron Works)

E. E. RIPPSTEIN*
(Laclede Steel Co.)†

JOHN F. SEIFRIED
(Ceco Steel Products Corp.)

With the increased use of more accurate methods of design of reinforced concrete structures in accordance with the ACI Code 318-41, it was evident that there is a need for better and more efficient detailing. This committee's report, "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures," released by the ACI Standards Committee for publication and discussion, was sent as a special publication (a large format made Journal publication impossible) to all members of record in July 1946‡. It was adopted by the 1948 convention as an ACI standard and this action was ratified by letter ballot canvassed June 29, 1948.

The response to the "proposed manual" was immediate. It is being bought in quantity by engineering offices and for college classroom use to an increasing extent. A second large printing (unrevised) was published in 1947 to meet the widespread demand. The book has been acclaimed as an indispensable tool of the drafting room and the class room.

Committee 323—Prestressed Reinforced Concrete

A. E. CUMMINGS, Chairman
(Raymond Concrete Pile Co., New York)

DOUGLAS E. PARSONS
(National Bureau of Standards)

IVAN ROSOV*
(New York City Tunnel Authority)

HERMAN SCHORER
(Borsari Tank Corp.)

H. B. ZACKRISON*
(Office, Chief of Engineers, Dept. of the Army)

This committee was assigned to the review of present knowledge of prestressed concrete, to develop design procedure and to recommend needed research.

Still other changes in committee personnel are indicated by reference to and following the order of committee listings in the ACI 1948 Directory.

Committee 212—Admixtures

WILLIS T. MORAN, Chairman
(Bureau of Reclamation, Denver)

Bruce E. Foster, National Bureau of Standards, has been added to the committee personnel; T. M. Kelly, formerly of Corps of Engineers, has resigned.

Committee 314—Rigid Frame Bridges

D. H. PLETTA, Chairman
(Virginia Polytechnic Institute, Blacksburg)

Douglas McHenry, Bureau of Reclamation, has been added to the committee; Arthur Sedgwick of Toronto, Ont., has resigned from its membership.

Committee 324—Precast Reinforced Concrete Structures

A. AMIRIKIAN, Chairman
(Bureau of Yards and Docks, Navy Dept., Washington, D. C.)

Nolan D. Mitchell, National Bureau of Standards, has accepted appointment to the committee.

Committee 609—Vibration of Concrete

HARMON S. MEISSNER, Chairman
(Bureau of Reclamation, Denver, Colo.)

H. L. FLODIN*, Secretary
(Portland Cement Assn.)

Bruce E. Foster, National Bureau of Standards, and Arthur A. Levison, Blaw-Knox Co. are two new members of the committee.

*Non-Member ACI.
†ACI Corporation Member.
‡Available to members at a special price of $1.75 and to non-members at the current published price of $3.00.

Committee 616—Recommended Practice for the Application of Paint to Concrete Surfaces

G. E. BURNETT, Chairman
(Bureau of Reclamation, Denver)

This committee is assigned to prepare recommended practices for the application of paints of different kinds to concrete surfaces. A report on portland cement-water paints was published (ACI JOURNAL June 1942) as information and for discussion. The next steps are: (1) submission of a recommended practice for use of portland cement-water paints, and (2) work on oil and resin paints.

Three new members appear on the roster of this committee which is nearly ready to report a proposed recommended practice for the use of portland cement-water paints. The new personnel: F. O. Anderegg, of the J. B. Pierce Foundation, W. Ellis Groben, a representative of U. S. Forest Service and G. S. Paxson, Oregon State Highway Department—all of whom have made studies in the field of the committee's assignment. Messrs. Anderegg and Paxon are ACI members.

Committee 621—Aggregate Selection, Preparation, Handling and Use

I. L. TYLER, Chairman
(Portland Cement Association, Chicago)

Arthur A. Levison, Blaw-Knox Co., is a new member of the Committee.

A.S.A. Sectional Committee (A42) on Plastering

Clayton L. Davis, Universal Atlas Cement Co., and J. W. McCoy, Lehigh Portland Cement Co are ACI's new representative and alternate on this A.S.A. committee.

A.S.A. Sectional Committee (A56) on Excavations and Foundations of the Building Code Correlating Committee

A. E. Cummings, Raymond Concrete Pile Co., has been named as ACI's representative, and Rene L. Bertin, consulting engineer, Long Island City, continues as alternate.

A.S.A. Sectional Committee (A2) Standard Specifications for Fire Tests of Building Construction and Materials

G. E. Troxell, University of California, has been made ACI representative.

A.S.A. Sectional Committee (A51) Building Code Requirements for Fire Protection and Fire Resistance

Rolland P. Cravens, Los Angeles Dept. of Bldgs. and Safety, has been named ACI representative.

A.S.T.M. Committee (E-5) Fire Tests of Materials and Construction

Raymond E. Davis, University of California, is now ACI's representative.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

PROTECTIVE COATING for concrete, masonry or steel is described in circulars available from Sika Chemical Corporation, Passaic, N. J. Sika-Seal is claimed to have excellent adhesion to wet or dry surfaces. It is not an emulsion. Test data listed in the leaflet support claims of protection given to concrete, mortar, wood, steel or other metal or non-metal surfaces. Two separate pamphlets are available—one particularly for architects and engineers and one for contractors and maintenance engineers. Products listed in another folder are Sika No. 2 and Sika No. 4a, liquid compounds which are mixed with cement to obtain initial set of the resulting mortar in as low as 20 seconds, the final set in as low as 30 seconds.

"FORM ENGINEERING" is the title of a 53-page catalog released by Williams Form Engineering Corp., Grand Rapids, Mich. The book describes available form hardware with photographs and detail drawings of actual installation. One 6-page section of the catalog is devoted entirely to design principles and detail drawings of small concrete retaining walls. A list of 44 free publications on concreting and concrete design is included.

"DESIGN MANUAL FOR AIRPORT PAVEMENTS," 32 pages, contains data on the use of welded wire fabric reinforcement in airport pave-

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Sika-Kote economically gives excellent protection to concrete and masonry structures on exterior or interior surfaces, above and below grade. Two easily applied coats prevent penetration of rain water or ground moisture even through cinder block walls.

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ment design. Closely following requirements of the United States Civil Aeronautics Administration, information given covers drainage systems, subbases and reinforced concrete pavements for runways, taxiways and aprons. Concisely presented material is supported by charts, tables, graphs and formulas. The book is available without cost from T. J. Kauer, managing director, Wire Reinforcement Institute, Inc., 1049 National Press Building, Washington 4, D. C.

PAVING MACHINES are described by the Chain Belt Co. in its new catalog No. 48-18 available on request from the company, 1600 W. Bruce Street, Milwaukee 4, Wis. The 28-page catalog illustrates major features of Rex single and double drum pavers with complete dimensions and specifications included, illustrated with photographs, line drawings and diagrammatic sketches in three colors.

"HORIZONTAL MOTO-MIXERS AND AGITATORS" is the title of Bulletin No. 48-9 also available from Chain Belt Co. Complete description of these truck mixers is given accompanied by on-the-job illustrations, and specifications for the 4½ yd and the 5½ yd mixer sizes.

"THE ABC'S OF SIMULATED SERVICE TESTING," Bulletin No. 291 of the Baldwin Locomotive Works, Philadelphia 42, Pa., has been prepared "for use by those who were self-admitted neophytes in the field" of life testing. It explains in basic terms (with cartoon illustrations) the whys and wherefores of simulated service testing, touching upon such topics as fatigue strength, endurance limit, constant amplitude machine and constant force machine.

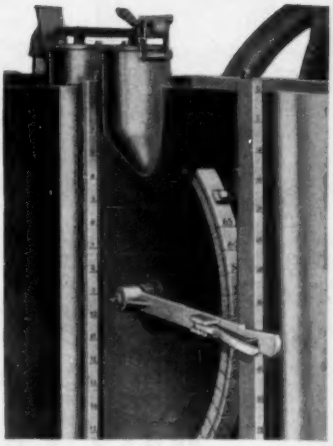
FATIGUE AND SIMULATED SERVICE testing by means of the Sonntag Universal fatigue and simulated service testing machine, is described by the Baldwin Locomotive Works in Bulletin 257. The brochure pictures the machine photographically and diagramatically, including a complete table of specifications.

CONVEYORS built in 8, 12, 16 and 20-ft sections with belt widths varying from 18 to 42 in. are described in a catalog available from the New Holland Manufacturing Co., Mountville, Pa.

STONFIL is the trade name of a "cement base water seal" which is said to work entirely by penetration and expansion, and to be applicable as a permanent seal for brick, unglazed tile, stucco, concrete, stone or pre-cast blocks. Stonfil is available in natural gray or a white which may be tinted to any of several shades with special coloring material. Details are available from the Ross Industrial Products Co., 3908 Olive Street, St. Louis, Mo.

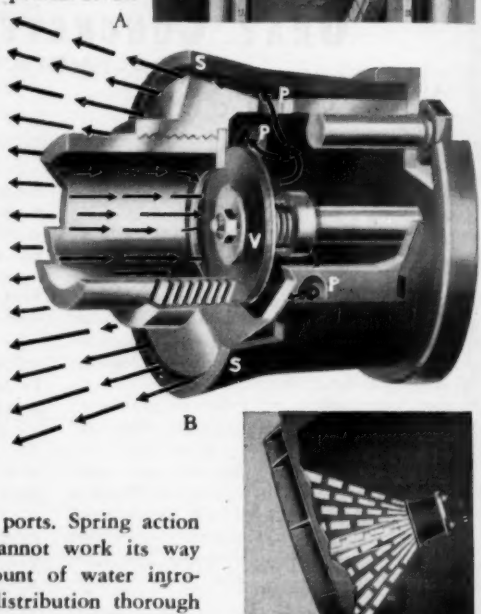
Measurement and distribution of water ... All-important in high quality concrete

Jaeger Truck Mixers incorporate many exclusive features for close control of amount of water introduced into transit-mixed concrete, for complete and efficient distribution of that water and absolute dependability of the supply and distribution system.



On all Jaeger Truck Mixers the mix-water tank (A) is equipped with an accurately graduated gauge which shows amount of water in the tank. Pre-setting amount of water to be introduced into the mix is facilitated by a large, calibrated quadrant and self-locking pointer-lever. A 1½-inch Jaeger Sure-Prime Pump draws water from the tank through an intake pipe which is accurately set at correct level by setting the pointer-lever. The narrow and deep tank increases accuracy of intake over old-style broad and shallow tanks.

A



B

The pump delivers water at 60 gallons per minute and 20 pounds pressure to a jet (B) in the center of the mixing drum (C). Pressure opens the jet valve (V). Water enters the jet chamber and escapes through ports (P) distending the tight-fitting sleeve (S). Water emits from the open end of the sleeve in a circular, penetrating stream directed into the mass of material at the lower end of the drum. When water pressure is relieved, the rubber sleeve immediately contracts, sealing the bell end of the jet and its chamber ports. Spring action closes valve (V). Thus, material cannot work its way into sleeve, ports or chamber. Amount of water introduced has been exact, input fast, distribution thorough and the system well protected against clogging and grouting.

C

Complete information on Jaeger Truck Mixers is yours for the asking.

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226 N. La Salle St. Chicago 1, Ill.
235 American Life Bldg. Birmingham 1, Ala.

ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting ACI current Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all ACI Standards except (ACI 315-48) in one book, $2.00 per copy—to ACI Members, $1.25.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to Members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel; "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention, and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy; $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

For further information about ACI Membership and Publications (including pamphlets presenting synopses of recent ACI papers and reports) address:

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REMINDER

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THE TURN OF SUMMER into fall is Nature's most poignant reminder of another year gone by.

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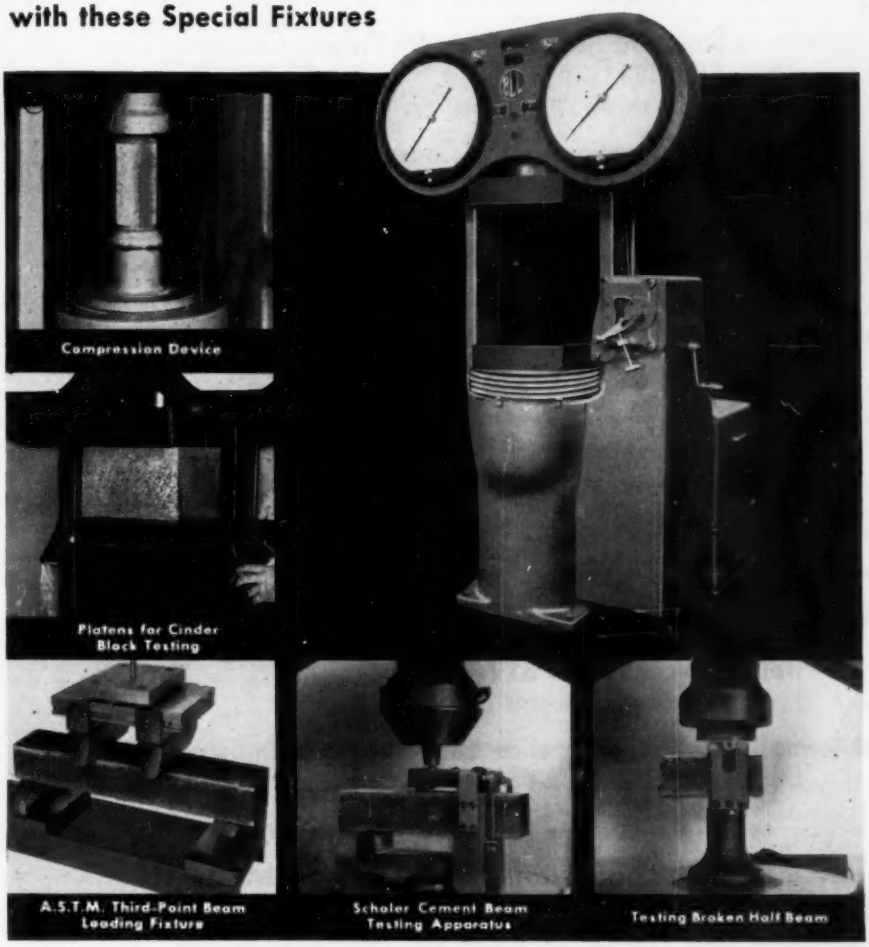
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Current ACI Standards

Building Code Requirements for Reinforced Concrete (ACI 318-47)
64 pages in covers: 50 cents per copy (40 cents to ACI Members)

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)
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Specifications for Cast Stone (ACI 704-44)
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Specifications for Concrete Pavements and Bases (ACI 617-44)
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Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)
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Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)
4 pages: 35 cents per copy

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)
A publication of large format, bound to lie flat.
55 pages: $3.00 per copy ($1.75 to ACI Members)

Recommended Practice for Winter Concreting Methods (ACI 604-48)
20 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recent Proposed ACI Standards

The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces
Reported by Committee 616 as information and for discussion only. 20 pages, 35 cents per copy (Reprint from ACI JOURNAL, June 1942)

Proposed Recommended Stresses for Unreinforced Concrete
Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Priced 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size 3/8 in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

NEW SYNOPSES PAMPHLET

Now available for distribution to those who ask for it is a new 8-page pamphlet reprinting the synopses of all ACI papers and reports published in Journal V. 19—Sept. 1947 to June 1948.

Those papers with concluding discussion and index to appear in the December 1948 Journal, Part II will comprise the complete Proceedings V. 44. Just ask ACI, 717 New Center Building, Detroit 2, Michigan for the new synopses pamphlet.

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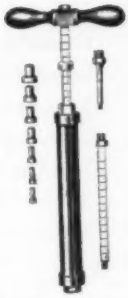

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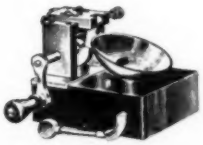

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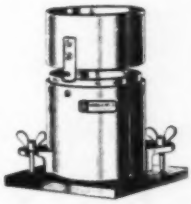

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DISCUSSION

Discussion closes January 1, 1949

Sept. Jl. '48

Recommended Practice for Winter Concreting Methods (ACI 604-48)—Report of Commitee 604

Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

Discussion closes February 1, 1949

Oct. Jl. '48

Lattice Analogy in Concrete Design—Douglas McHenry

Strength of Precast Concrete Floor Joists—Jacob Feld

Effect of Entrained Air on Concretes Made With So-Called "Sand-Gravel" Aggregates—Paul Klieger

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture—P. H. Petersen

Discussion closes March 1, 1949

Reinforced Concrete Wall and Column Footings—Frank E. Richart

The use of precast reinforced concrete is no new idea but there is a new interest in it. Structural frame members of reinforced concrete can now be selected (knowing load and span requirements) from a manufacturer's handbook in a similar manner as for structural steel. Form work, commonly regarded as a necessary evil, is eliminated as the precast structural frame is erected by motor-crane. Mr. Grafflin presented his ideas on this advanced method of construction at the Denver convention—last man on a long evening program—and held his audience not only for his formal paper but for an added period (approved by a show of hands) when he offered more pictures of his work in progress.

Cemenstone Precast Construction*

By A. C. GRAFFLIN†

SYNOPSIS

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

INTRODUCTION

The building industry needs standardization, simplification and mechanization of its processes.

For most ordinary building needs—that is, for structures of four stories or less, which comprise more than 90 percent of all buildings—precast concrete construction offers a solution to this difficult problem and indicates a path which, if followed logically, will lead to better buildings for less cost. It is not suggested that the method described here and called "Cemenstone" is the only possible way to achieve better construction for average needs. But it is a method that has been thoroughly proved in the field against keen competition and has produced satisfactory, low-cost, fire-resistant buildings quickly.

A great advantage of steel and timber shapes for framing purposes has been that they are available in a wide range of individual sizes, which may be handled easily and assembled into complete structures. Until now, a disadvantage of concrete has been that it has had to be handled in the field as a semi-fluid material, usually requiring special structural design, form-work and pouring for its structural development and use. This is generally expensive and time-consuming.

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Precast Cemenstone construction combines all of the advantages of individual structural shapes which may be easily handled and assembled into completely framed, covered and enclosed structures, with the added advantage that all elements are made of fireproof concrete.

THE SYSTEM

First, the structural shapes required for the framing and, if desired, the roofing, flooring and side wall panels for all ordinary structural needs, have been reduced to the fewest possible number, about ten in all. For example, a one-story structure may be completely framed and enclosed by the use of but four basic members, regardless of their individual sizes —columns, girders, bracing roof panels, and side wall panels.

These shapes have been standardized and simplified in design, but are extremely flexible in sizes to meet all requirements of load and length within their standard ranges of up to 50 ft for framing members and 25 ft for roof and side wall panels.

By means of the carefully engineered and standardized connections welded to the primary reinforcing, all structural shapes, regardless of individual size, are designed to be accurately assembled to give framing at all times sufficiently rigid to meet every engineering requirement. Fig. 1 shows typical connection details.

Complete load tables based on simple spans have been developed for the various shapes in the same way as for steel and timber, which greatly

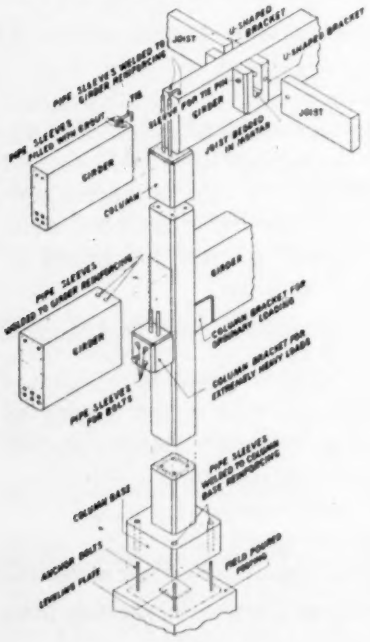


Fig. 1—Isometric view of typical column showing connection details

simplify the designer's problems. A portion of these tables is shown in Fig. 2. By reference to the tables the sizes, weights and capacities per lineal foot for each shape are quickly determined with a minimum of computations; also by means of standard details manpower in the detailing of structures is held to a minimum. An important feature of these shapes is that in manufacture there is combined in one operation, in one plant and before erection, all of the requirements of fabrication and fire protection.

In addition to structural framing members two other standard members have been developed which will integrate with this structural system, or which may be used independently with other types of framing. One is a large roof panel slab which combines in one member all of the functions of joists, bracing beams and roof decking. This panel slab is made in two types—one with fixed dimensions of 5 ft x 20 ft, and the other with varying dimensions in width from 4 ft to 6 ft, and in length from 18 ft to 25 ft, and in live load capacity from 30 to 100 lb per sq ft. The advantages of having in one member a true and unbroken surface of 100 sq ft or more of area as compared to the use of smaller sections, are obvious. These panels are bolted into the girders to provide all necessary bracing.

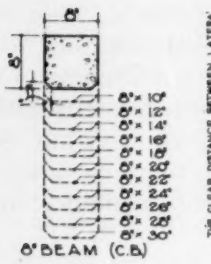
An added advantage of the use of these panels is that the building is covered as erection proceeds, so the ground floor may be filled and poured from the day erection begins, instead of having to wait until the entire building is framed and then covered.

The second new member is a large wall panel for use in place of typical built-up masonry walls. These panels are usually 4 in. thick and may be made up to 8 ft high by 25 ft long. They are made of the same high quality concrete, which is virtually impervious to moisture penetration, and provide weatherproof, unbroken surfaces of large area.

They are fabricated in the plant while the framing is being made, and erected at the same time as the framing. The great saving in time is obvious. They are lower in cost than brick or tile, and greatly simplify erection problems, providing modern, pleasing, weatherproof exterior finishes with fire wall protection.

Because they are completely self-supporting, they have been used several times as grade beams suspended between the columns, eliminating the usual poured wall foundation. Their use eliminates, for structural purposes, any need for sills, lintels or copings.

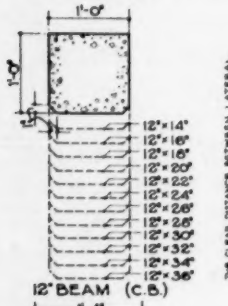
An important feature of these panels is the flexible or "floating" connections which have been developed, which permit their attachment to all types of framing in such a manner as to absorb all movement of the panels due to extreme changes in temperature, or to settlement or other causes. In addition, effective methods have been developed for both



ALLOWABLE UNIFORM LOADS IN POUNDS PER LINEAL FOOT

THE CLEAR DISTANCE BETWEEN LATERAL SUPPORTS SHALL NOT EXCEED 21.33 FT.

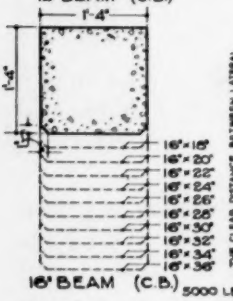
SIZE \ SPAN	8'	9'	10'	11'	12'	13'	14'	15'	16'	17'	18'	19'	20'
8"× 8"	1110	861	688	557	467	380	318	269	229				
8"× 10"	2008	1565	1257	1024	847	712	601	512	441	381	331	288	252
8"× 12"	3167	2476	1994	1650	1353	1140	966	830	719	625	547	480	424
8"× 14"	4586	3593	2899	2375	1977	1669	1422	1225	1063	927	816	719	636
8"× 16"	6234	4879	3935	3221	2679	2281	1924	1653	1436	1251	1099	968	857
8"× 18"	7984	6209	4526	3706	3084	2605	2217	1906	1656	1444	1270	1119	992
8"× 20"	8723	7242	5850	4797	3996	3361	2864	2485	2164	1892	1666	1474	1311
8"× 22"	9756	8647	7341	6025	5027	4256	3636	3137	2737	2397	2117	1875	1671
8"× 24"	10789	9564	8584	7392	6173	5231	4474	3864	3375	2959	2618	2322	2073
8"× 26"	11823	10481	9408	8530	7436	6306	5398	4666	4079	3581	3171	2817	2517
8"× 28"	12856	11398	10231	9276	8481	7479	6406	5542	4846	4259	3775	3356	3005
8"× 30"	13889	12314	11054	10023	9164	8437	7500	6492	5685	4996	4431	3943	3530



ALLOWABLE UNIFORM LOADS IN POUNDS PER LINEAL FOOT

THE CLEAR DISTANCE BETWEEN LATERAL SUPPORTS SHALL NOT EXCEED 32 FT.

SIZE \ SPAN	8'	9'	10'	11'	12'	13'	14'	15'	16'	17'	18'	19'	20'
12"× 12"	4748	3714	2991	2445	2030	1710	1452	1245	1078	937	821	720	635
12"× 14"	6881	5389	4348	3561	2964	2503	2132	1833	1594	1390	1223	1078	954
12"× 16"	9366	7337	5921	4849	4037	3409	2904	2498	2171	1894	1666	1469	1305
12"× 18"	12283	9630	7780	6381	5320	4499	3840	3309	2885	2521	2224	1966	1749
12"× 20"	13102	10880	8792	7212	6014	5088	4344	3745	3263	2855	2519	2229	1983
12"× 22"	14652	12909	11028	9085	7589	6402	5472	4724	4123	3612	3193	2830	2524
12"× 24"	16201	14364	12894	11106	9278	7865	6729	5814	5080	4457	3944	3501	3127
12"× 26"	17751	15739	14129	12812	11171	9476	8115	7017	6136	5388	4774	4242	3793
12"× 28"	19302	17114	15364	13931	12739	11236	9626	8331	7290	6407	5680	5052	4522
12"× 30"	20852	18490	16600	15054	13765	12675	11268	9757	8543	7513	6666	5933	5315
12"× 32"	22402	19864	17834	16174	14789	13618	12614	11293	9893	8704	7727	6881	6168
12"× 34"	23952	21240	19070	17295	15815	14563	13490	12560	11343	9984	8868	7902	7087
12"× 36"	25502	22614	20304	18414	16839	15506	14364	13374	12508	11350	10084	8989	8066



ALLOWABLE UNIFORM LOADS IN POUNDS PER LINEAL FOOT

THE CLEAR DISTANCE BETWEEN LATERAL SUPPORTS SHALL NOT EXCEED 42.66 FT.

SIZE \ SPAN	8'	9'	10'	11'	12'	13'	14'	15'	16'	17'	18'	19'	20'
16"× 16"	12504	9796	7906	6478	5384	4538	3864	3342	2908	2538	2234	1972	1750
16"× 18"	16393	12852	10365	8520	7105	6010	5132	4424	3856	3375	2977	2633	2344
16"× 20"	17481	14519	11735	9628	8031	6796	5804	5006	4363	3819	3371	2984	2656
16"× 22"	19548	17351	14716	12086	10091	8549	7309	6311	5510	4829	4270	3786	3378
16"× 24"	21614	19164	17204	14820	12383	10499	8984	7764	6785	5955	5271	4680	4181
16"× 26"	23681	20997	18851	17095	14907	12647	10850	9368	8194	7196	6378	5668	5071
16"× 28"	25748	22831	20498	18568	16998	14994	12847	11121	9732	8555	7586	6749	6042
16"× 30"	27815	24665	22145	20063	18365	16911	15036	13021	11402	10029	8900	7922	7098
16"× 32"	29882	26499	23792	21577	19732	18170	16832	15071	13204	11619	10318	9188	8237
16"× 34"	31948	28331	25438	23071	21098	19429	17998	16758	15135	13323	11835	10547	9463
16"× 36"	34015	30165	27085	24565	22465	20686	19165	17845	16690	15146	13458	11998	10761

NOTE: DESIGN BASED ON THE AMERICAN CONCRETE INSTITUTE CODE 1941, 5000 LB. PER SQ. INCH CONCRETE IN COMPRESSION, AND INTERMEDIATE GRADE STEEL.

Fig. 2—Partial reproduction of typical design tables for beams and

vertical and horizontal joints whereby the seal between panels is always maintained, even under extreme conditions.

DESIGN

The design of all members conforms to the requirements of "Building Regulations for Reinforced Concrete" (ACI 318-47) and when required can be modified to conform to other special codes. Normally all beams-action members are designed as simple beams with working stresses of 2,250 psi for concrete and 20,000 psi for steel. Intermediate or hard grade steels only are used for reinforcing. The specially designed connections (Fig. 3 and 4) provide adequate stiffness in themselves for most building designs. Where special conditions of rigidity or continuity are desired, they may be obtained by modifying the connection

WEIGHTS OF THE BEAMS OR GIRDERS HAVE BEEN DEDUCTED.

34'	35'	36'	37'	38'	39'	40'	41'	42'	43'	44'	45'	46'	47'	48'	49'	50'	SPAN / SIZE	WEIGHT PER FT. OF BEAMS
																	8" × 8"	66 LBS.
																	8" × 10"	83 "
																	8" × 12"	100 "
																	8" × 14"	116 "
																	8" × 16"	133 "
222																	8" × 18"	150 "
321	292	265	240	217													8" × 20"	166 "
435	398	365	334	305	278	254	231										8" × 22"	183
563	516	477	439	404	371	341	313	288	264	241							8" × 24"	200 "
706	652	603	557	515	476	440	407	376	347	321	295	272	250	229			8" × 26"	216 "
865	799	741	688	636	591	549	510	473	439	408	378	350	324	299	275	255	8" × 28"	233 "
1084	960	893	830	772	717	668	622	580	540	503	468	436	405	377	349	325	8" × 30"	250 "

THESE LOAD TABLES FOR THE 18" TO 36" DEPTHS ARE BASED ON THE WEIGHT OF THE GIRDERS WHOSE TWO SHELVES WEIGH 15 LBS EACH PER FOOT.
15 LBS. PER LIN. FT. ADDITIONAL MAY BE ADDED TO THE LOADS IN THE TABLES FOR THE END-GIRDERS AND 36 LBS. PER LIN. FT. ADDITIONAL MAY BE ADDED TO THE LOADS IN THE TABLES FOR 18" TO 36" DEPTH BEAMS.
TO OBTAIN WEIGHT OF GIRDERS ADD 36 LBS PER LIN. FT. TO THE GIVEN BEAM WEIGHTS.
" " " " END-GIRDERS " 15 " " " " " " " " "

34'	35'	36'	37'	38'	39'	40'	41'	42'	43'	44'	45'	46'	47'	48'	49'	50'	SPAN / SIZE	WEIGHT PER FT. OF BEAMS
																	12" × 12"	150 LBS.
218																	12" × 14"	175 "
297	266	239	214														12" × 16"	200 "
435	395	360	327	296	267	242	217										12" × 18"	225 "
499	455	415	378	343	310	281	254	229	205								12" × 20"	250 "
670	614	564	518	475	434	398	364	332	302	275	249	225	203				12" × 22"	275 "
862	794	733	677	624	574	530	488	449	413	380	348	319	291	265	240	218	12" × 24"	300 "
1076	995	922	854	791	731	678	628	581	538	498	460	425	392	360	330	304	12" × 26"	325 "
1312	1216	1130	1049	974	904	841	782	727	676	629	584	543	503	466	431	399	12" × 28"	350 "
1571	1459	1356	1264	1177	1094	1021	952	888	829	774	721	673	627	584	543	506	12" × 30"	374 "
1849	1720	1603	1495	1395	1300	1215	1136	1062	993	930	869	814	761	710	663	621	12" × 32"	400 "
2151	2003	1870	1747	1632	1523	1427	1336	1252	1173	1101	1031	968	907	850	796	747	12" × 34"	424 "
2473	2305	2155	2015	1885	1761	1652	1549	1454	1365	1282	1204	1132	1064	999	937	882	12" × 36"	450 "

NOTE: TO OBTAIN THE EQUIVALENT UNIFORMLY DISTRIBUTED LOAD — MIDDLE POINT LOADING: MULTIPLY THE SINGLE LOAD BY 2. EQUAL THIRD POINT LOADING: MULTIPLY ONE OF THE LOADS BY 2⅔.
IN THE BEAMS AND GIRDERS THE REINFORCING STEEL WILL BALANCE THE STRENGTH OF THE CONCRETE.

34'	35'	36'	37'	38'	39'	40'	41'	42'	43'	44'	45'	46'	47'	48'	49'	50'	SPAN / SIZE	WEIGHT PER FT. OF BEAMS
409	367	331	298	267	237	211											16" × 16"	266 LBS.
592	539	492	448	407	368	335	301	272									16" × 18"	300 "
676	619	566	516	470	426	387	351	318	286								16" × 20"	333 "
906	831	765	705	646	591	543	498	458	418	379	345	313					16" × 22"	366 "
1161	1071	989	915	844	777	719	663	611	562	519	476	437	400	368	332	305	16" × 24"	400 "
1447	1339	1242	1151	1067	987	916	849	787	730	676	626	579	535	492	452	417	16" × 26"	433 "
1762	1634	1519	1411	1311	1218	1134	1055	982	914	851	791	737	683	634	587	545	16" × 28"	466 "
2106	1957	1822	1697	1581	1470	1373	1281	1196	1117	1044	973	909	848	790	736	686	16" × 30"	499 "
2479	2307	2150	2007	1873	1747	1633	1528	1429	1337	1253	1172	1099	1028	960	897	841	16" × 32"	532 "
2879	2682	2505	2341	2187	2042	1914	1793	1681	1575	1479	1386	1302	1221	1144	1073	1007	16" × 34"	566 "
3310	3086	2886	2700	2526	2361	2216	2078	1951	1833	1722	1618	1522	1431	1345	1262	1189	16" × 36"	599

[THESE TABLES BASED UPON A CONCRETE WEIGHING 150 LBS. PER CU. FT.]

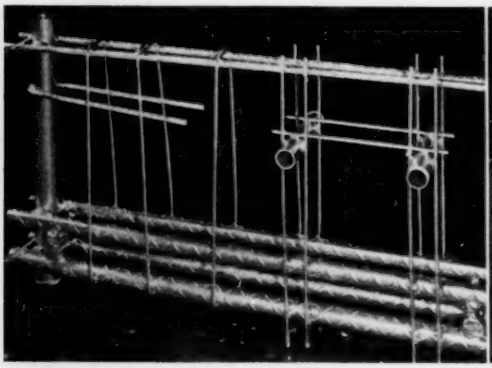
girders. Similar tables have been developed for columns, joists and panels

designs to include welding the primary reinforcement of horizontal members through or across the columns.

All members are designed for safe handling during erection as well as for their design loads. For example, a 60 ft column which is cast and handled horizontally until erected, is designed as both a beam and a column. Wall panels up to 8 ft high are cast horizontally and are designed for safe handling without supplementary framing support until erected. Wall panels are also designed for column action to make them self supporting to full height of the building.

MANUFACTURE

The standardization in Cemenstone shapes is achieved in their manufacture. Each column, girder, beam or joist varies in modules of 4 in.



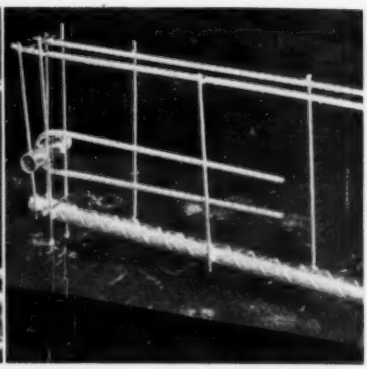


Fig. 3 (left)—Reinforcing for beam or girder connection. Fig. 4 (right)—Reinforcing for joist connection. Every part of the reinforcing and connections for all structural shapes are welded into a rigid cage before casting. The tips of all spreader or supporting bars which reach the surface of the concrete and would be exposed in any way are tipped with Monel metal to prevent rusting.

in width, from 4 in. to 16 in., and in modules of 2 in. in depth, from 8 in. to 36 in. The larger forms for columns, girders and beams are 60 ft long, which permits the casting of single members up to that length, or of two or three members with a total length of not over 60 ft at one time. Length variations may be specified to the nearest ¼ in. if necessary.

The reinforcing for members is welded into units or cages; connection sleeves to receive dowels and bolts in the field erection are accurately located and welded in place to complete the reinforcing assembly which is then ready for the forms. Fig. 5 shows such a unit for a 53-ft outdoor theatre column and the completed member.

The standardized, specially developed forms used for the various structural elements are, in most cases, cored with concrete. Pipes are embedded in these on the radiant heat principle through which steam or heated water is circulated to accelerate the set of the concrete and to balance the internal heat developed by the Type III high-early strength cement. This permits the early stripping of the forms, and their removal to new pallets for the casting of new members. Meanwhile the original members remain on the pallets under damp covers kept continuously saturated with heated vaporized water until they are sufficiently set to permit safe handling.

Initial moist curing is four to eight hours for most members. They are then removed to the curing chambers where they are moist cured for three days by low pressure steam or vaporized heated water. This method of forming and curing greatly reduces the usual problems of shrinkage, warping and the tendency toward incipient cracks and crazing.

The concrete used for structural shapes is a very rich mix—8½ sacks per cu yd—placed with an average slump of 2 in., carefully and heavily

vibrated both internally and externally to produce a dense hard quality which is virtually waterproof. It developes a compressive strength of 5,000 psi within three days and 6000 psi within seven days. Standard aggregates are used throughout. The consistent development of such high quality concrete under ideal shop control conditions permits the use of smaller size structural members than would otherwise be required.

To achieve better finishes where architectural values and effects are important, panels are cast against muslin or other textured fabrics which remain in place on the panels until after they are erected to protect them against soiling from handling. Not only does this method provide a pleasing finish to the surface but, equally important, it overcomes any tendency of the surface to craze or check in weathering.

ERECTION

One of the great advantages of this method of construction is speed of erection. Footings are cast in place to receive the precast elements

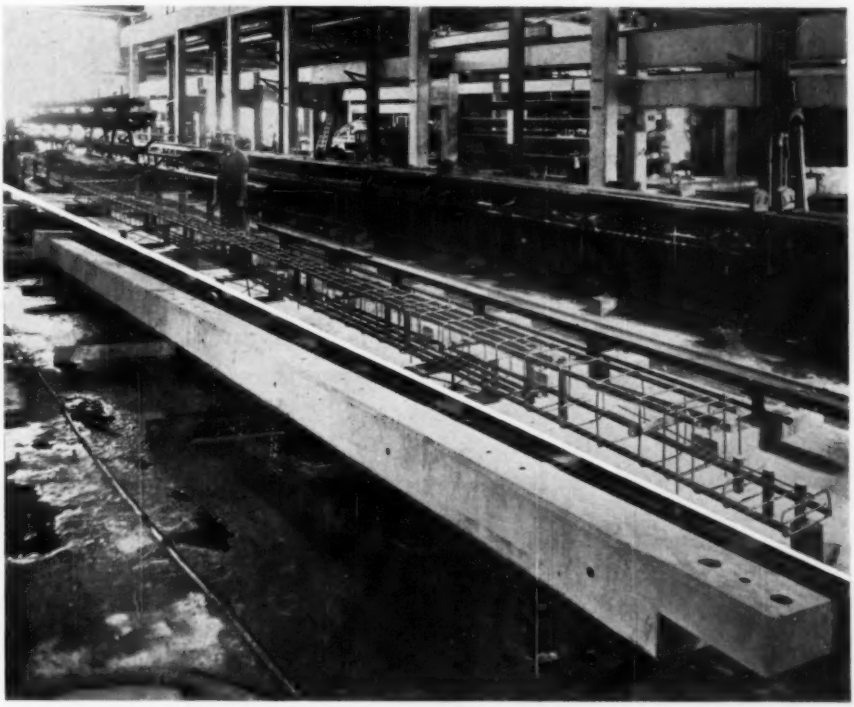


Fig. 5—Welded reinforcing "cage" (top) developed in typical manner for use in casting the 53 ft long precast column (bottom) which was used for outdoor motion picture screen. Column is of similar size and length to those used in the precast four-story building shown in Fig. 11. In the background are shown the steel forms used for casting, a typical pallet, and some typical precast framing in use.

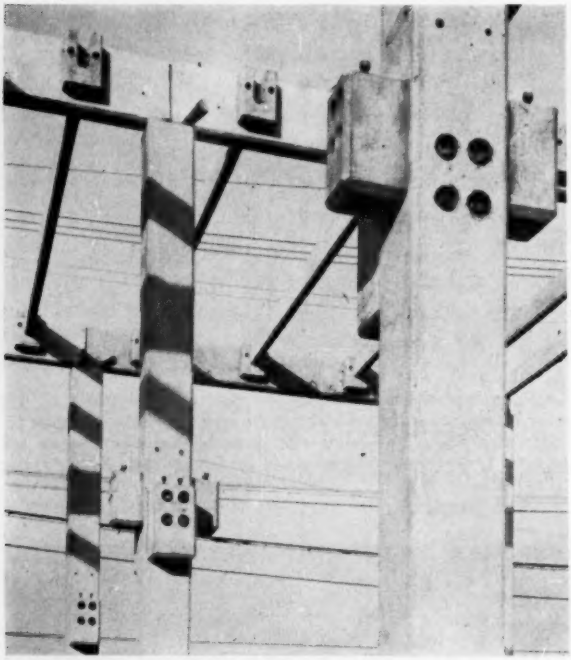


Fig. 6—A close-up showing some typical connection methods for a heavy duty structure. All brackets may either be attached at the plant or in the field. After being bolted to the columns and sealed to prevent any loosening of the nuts due to vibration, the counter-sunk openings shown are all capped with concrete for fire protection. Neither U-brackets nor block brackets as shown are used where lighter loads are involved.

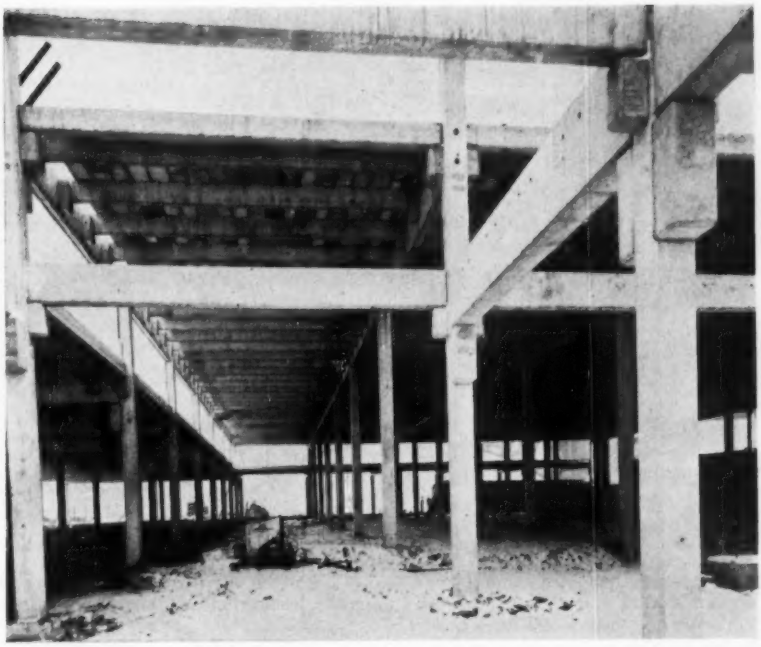
which are usually hauled to the site by truck and easily erected by a motor crane. All connections are specially designed to join members with bolts or dowels grouted into sleeves in the field. Accuracy in manufacture permit clearances of as low as $\frac{1}{8}$ in. between members (After the erection of more than 40 buildings there has been no instance of a failure to fit and assemble accurately all members of a building). A leveling plate is provided on top of footings to permit plumbing columns with ease and accuracy before finally grouting them securely into place.

EXAMPLES

Most of these buildings have been of one and two stories; examples are shown in Fig. 6 to 10 in various stages of completion. Fig. 11 shows the frame of a four-story office structure being erected for the Blaw-Knox Co. at Blawnox, Pa. This building was developed and designed by the engineering firm of Hunting, Larson and Dunnells of Pittsburgh, Pa., using Cemenstone tables for the selection of the structural shapes in the various sizes required. Continuous columns over 50 ft long were used. As far as is known, this is the first wholly precast four-story structure ever built using continuous columns.

COSTS AND MARKETING

Comparative costs of this type of construction will vary, of course, in different localities. In the Pittsburgh district, the principal home



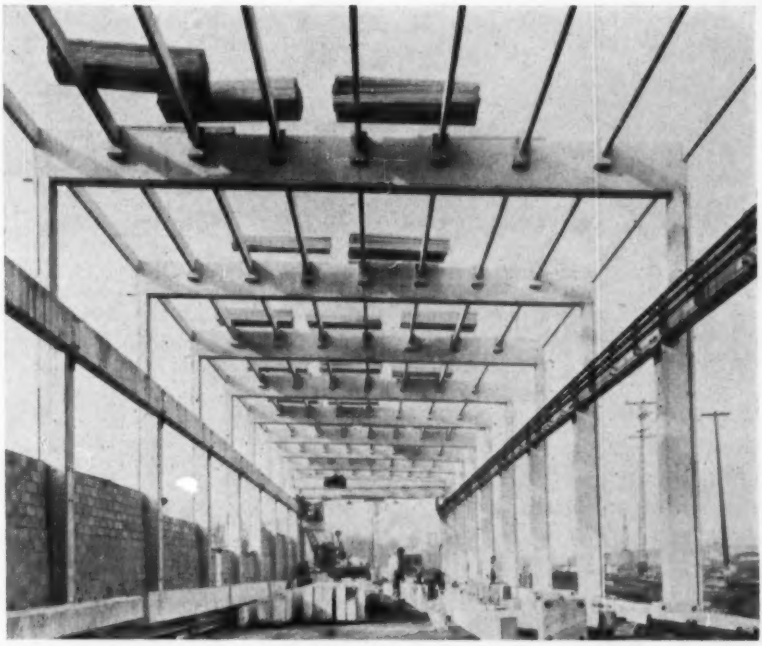


Fig. 7 (top)—Typical section of a building designed for heavy duty. The second floor live load of the two-story part of this structure is 300 lbs. Fig. 8 (bottom)—One-story structure with uniform bays. Note that special connections, as in this instance for crane rails, may be provided easily.

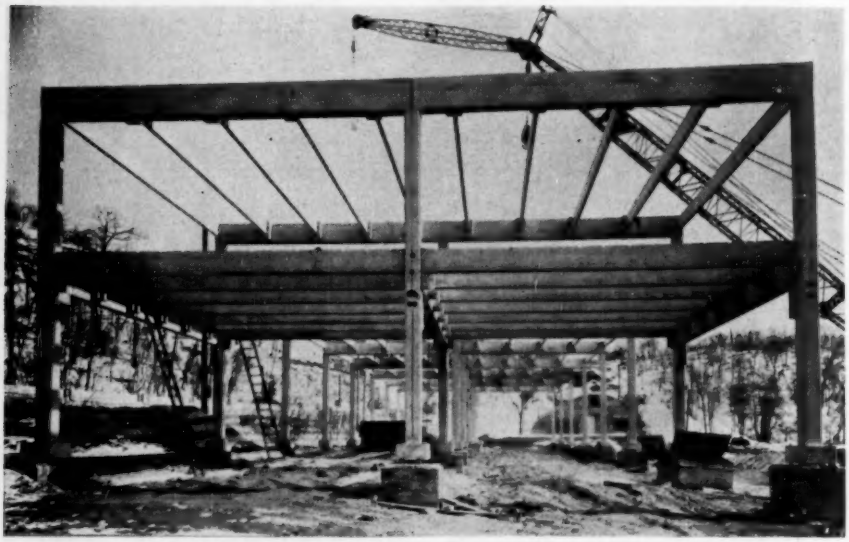


Fig. 9—A combination of a standard one and two-story structure which in varying dimensions up to 100,000 sq ft of floor area, may normally be fabricated, delivered, and erected at any time of the year in 60 days or less.

Fig. 10—Typical one-story building framed, roofed, and enclosed with precast structural shapes and side wall panels. The panels slot into the columns where architectural emphasis is desired as in this case, or they may be run across and attached to the face of the columns. The panels in this case are also used as grade beams, eliminating the necessity of poured wall foundations between the columns.

of structural steel, where competitive conditions are probably sharper than anywhere else in the country and where most Cemenstone buildings have been erected, the experience to date has been that for most average requirements this construction is directly comparable in cost with structural steel unprotected, and at least 20 percent less than steel fireproofed, or poured-in-place concrete. This difference is, of course, accentuated during the winter months when protection is required in the field for other types of construction.

Obviously, there is a practical limit in the distances to which any precast structural shapes can be shipped and erected. It has been found

that this construction can be handled most efficiently and economically where delivery from the plant to the site can be made by trucks, and erection undertaken directly from the trucks. It is believed that the maximum efficient operating distance from the plant is probably within 200 miles, although this material has actually been shipped by rail to greater distances and erected at competitive cost against comparable construction.

CONCLUSION

The design and development of this system over the past twenty years has been in the hands of a small group and all fabrication confined to the parent plant in Pittsburgh. It has been concluded, however, that the fabrication and use of Cemenstone shapes, to be most effective, should be undertaken on a local basis, and therefore a plan for local fabrication under license* has been developed.

*The methods, shapes, and connections are fully covered with more than 60 patents issued and pending, which are also registered in many foreign countries.

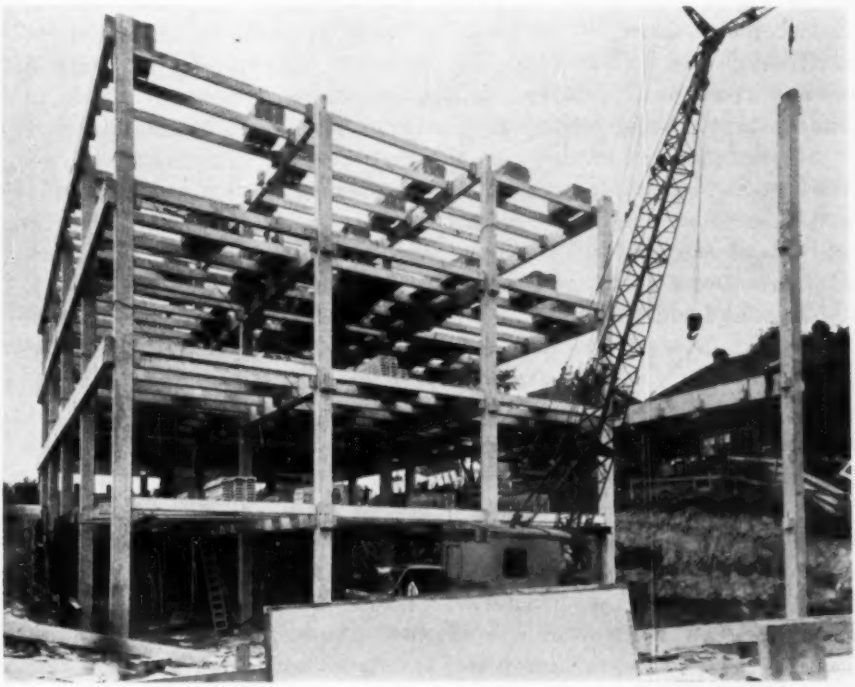


Fig. 11—Four-story wholly precast office building. The continuous columns are over 50 ft high. The rigidity and accuracy of this system is indicated by the fact that the columns are erected without guy wires. With maximum tolerances of ¼ in. every girder and joist at each successive floor has dropped perfectly into place without adjustment. Floor live loads vary from 60 lb to 125 lb, and the structure will be used as an office building, with some fairly heavy business machines on the second floor.

Germany has generally been supposed to have made further progress in precast construction than any other country. The Joint Intelligence Objectives Agency of the United States recently completed a study of German methods, and quote in their report No. 19 the most outstanding authority in Germany on precast concrete, Dr. I. Hubert Rusch, as follows:

> "Three important problems remain concerning the development of precast concrete members in the building industry. They are:
> 1. Standardization of structural members for mass production precasting.
> 2. Reduction of construction costs by improvements in the precasting process and organization of the industry.
> 3. Development of connections between the structural members."

These are the problems that have been given the closest study during more than twenty years of development, and which have been effectively solved. A careful study of the final Joint Intelligence Agency report No. 1123 on the precast concrete products industry in Germany, written by Mack A. Arnold and Otto L. Formigli and published in June 1947, indicates little progress or development in Germany or elsewhere toward the practical solution of these problems, comparable to that which has been achieved here.

Although the effective solution of many construction problems for average needs has been accomplished, connections, processes and designs are constantly being perfected. This is expected to continue. Research is now in progress in the use of higher strength concrete and higher yield point steel, which means still smaller sections; the use of hollow and tapered sections; the use of pre-stressing for longer span girders and beams—and in other features of precast construction which show high promise of further efficiencies and economies.

Newer and wider uses for precast shapes and construction are being developed: multi-story buildings, precast grandstands, apartments, houses, schools, hospitals, small bridges, airport and utility buildings, and other structures. Recently an architect designed a wholly precast outdoor motion picture screen, using standard shapes throughout, with columns over 50 ft high. This has been erected and is in successful use today. An enterprising engineer in Maine has developed an apparently practical design for a wholly precast outdoor swimming pool, using standard shapes throughout. It would seem that the field of usefulness for precast construction is only beginning, and has unlimited possibilities.

This Cemenstone method of construction provides a practical way for greatly broadening the effective use of concrete, with all of its advantages. It greatly simplifies many design and field problems of engineers, architects, and contractors. It requires less equipment, personnel, and time to erect than comparable types of construction. It is a long step ahead in the field of construction because of its simplicity, high quality, great flexibility, economy, and savings in time in both fabrication and erection.

Discussion of a paper by A. C. Grafflin:

Cemenstone Precast Construction*

By CHARLES W. DOHN and AUTHOR

By CHARLES W. DOHN†

Completely precast concrete construction is perhaps the greatest advance in masonry construction since mass production of brick. Mr. Grafflin and his company are to be commended for having developed an admirable precast reinforced concrete construction which compares well with fireproofed structural steel, particularly for buildings of not more than four stories. The design tables presented are excellent and mark a firm step in the direction of standardization of precast construction the form of which ACI Committee 324—Precast Reinforced Concrete Structures, might well consider in establishing standard recommendations for cross sections, spans and loads. Standardization of such units aid in their design, use and mass production by universal machinery and methods. Modular dimensioning and whatever ACI specifications might be applicable or proposed therefore should be adhered to.

Would lateral pressures and torsional tower strains cause appreciable harm over a long period in a building so constructed? Of course, for the four-story limited-height building, such effects would not be large and connections are simply designed. Can moment connections for this type structure be standardized? This would seem to be important, as this very question—standardization of moment connections for beams—is still a problem in the design of welded steel building frames. Has this type building been considered for designs in areas where seismic disturbances are a factor in design? Could such a building be designed adequately and what might be some recommendations or salient features for such a design?

It is conceivable that buildings taller than four stories could be constructed by somewhat similar methods if sufficient moment connections or braces are provided to handle wind and sway stresses. Complete precast construction of tall buildings offers a worthwhile challenge to the

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†Professional Engineer, New York, N. Y.

engineer since time saved in erection, handling of concrete and reinforcing material, and forms and props would be almost proportional to the height of the building. Long delays now hamper tall reinforced concrete building construction. For instance, several days must be allowed between story height pours, even with high early strength concretes and the most efficient concrete handling methods. For a tall building, especially, this time represents wasted overhead for contractor's plant and equipment, storage problems and lost rentals and high on-the-job labor costs. In view of all this, precast reinforced concrete would compare well with fireproofed steel construction for apartment and office buildings and well as other uses up to, say, fifteen stories or more in height. Further adoption of lightweight aggregates and prestressing techniques would increase their amenabilities even more.

AUTHOR'S CLOSURE

The questions Mr. Dohn raises in his excellent discussion are pertinent and searching.

Precast construction is still in its infancy. In the Cemenstone method, considerable progress has been made toward the simplification of construction processes, which is so essential to the building industry. There are some logical next steps clearly foreseeable, some of which are covered by Mr. Dohn's questions and which are now in process of development.

As at least 90 percent of all buildings are from one to four stories in height, the primary effort so far has been in that field. As Mr. Dohn has pointed out, lateral pressure and torsional tower strains in such buildings are very small. In more than fifty completely framed structures, up to over 50 ft in height, erected during the past three or more years, no such strains or pressures sufficient to cause the slightest effect upon these structures have been observed, and careful checks have been made to determine such possibilities.

Considerable work has been done to perfect connections which will develop a moment, and this problem has been pretty well solved. Rigidity to the extent required is obtained by adding additional top steel in lateral members, which is welded through or across the columns, and this steel is then covered and protected with concrete. All connections are standard and a few different ones serve for all types of members. This procedure has met every structural requirement.

Considerable thought has also been given to effective column splices for vertical precast members for multi-story buildings. It seems apparent that satisfactory connections can be provided to fully develop the columns for both direct load and moment, although such erection has not actually been done so far in Cemenstone construction.

It is true that in precast construction, the same as with poured-in-place concrete, columns are generally larger than steel columns for higher buildings. The enormous advantages in time-saving, lower cost, and maintenance may well offset this slight disadvantage.

In average low buildings, weight is not nearly as important as in higher ones. Economically, the added cost of good lightweight aggregates for main structural members can be questioned, except in a few cases where weight is all-important and sizes of members less so. There is a considerable loss of strength through their use, and higher cost and increased section sizes naturally are involved. Of course, in higher buildings, the use of lightweight aggregates, especially for secondary members, would make a considerable difference in column and footing loads.

Unquestionably, prestressing of concrete is the next logical step ahead in precast construction. A great deal of experimental work has been done in this field and it shows definite promise. The combination of prestressing with mass-produced standardized-shape members promises enormous advantages in building. However, before prestressing can be generally used, far more data will have to be available to convince building departments in many cities to add it to their codes.

No Cemenstone buildings have so far been definitely designed for use in any earthquake areas. There is no reason why such structures could not be developed, using standardized shapes and connections, if all joints and connections provide sufficient rigidity as well as flexibility. This factor of flexibility, properly developed, might be highly important. About two years ago, a prominent engineer who has successfully designed several buildings in earthquake areas, expressed the opinion that Cemenstone construction, because of a certain flexibility in its connections and the very solid nature of its foundation, might provide a possibly higher degree of resistance to earthquakes, especially up to forces 5 or 6, than any all-rigid poured-in-place construction. This may be true and seems logical but has not yet been subjected to any specific tests.

There is, in my judgment, one fundamental principle in all precast design and construction which should never be lost sight of. Indeed, I think it is its only justification for existence. That principle is—"If it isn't simple, it isn't right," and to this might be added, "And it isn't good precast design either." In other words, the very essence of good precast construction is that it provides a simpler, quicker, more economical method of handling concrete and for achieving better and more durable construction. That has been the guiding principle of everything Cemenstone has endeavored to develop in its method.

Air-entrainment in concrete continues as an important pre-occupation of many students of increased durability. Mr. Craven's paper is one of two in this issue (see p. 217) both having a common concern as to sand size—as one of the numerous factors influencing an optimum balance between strength loss and gain in durability.

Sand Grading Influence on Air Entrainment in Concrete*

By M. A. CRAVENt

SYNOPSIS

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30–No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30–No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

PREPARATION OF CONCRETE MIXES

The tests as herein described were conducted at the Engineering Materials Laboratory of the University of California, under the direction of Prof. Raymond E. Davis. Due to the short time available for the program, tests had to be confined to a limited number of conditions. One cement and two nominal cement factors were used and all tests were made at a slump of 3.5 in. Three air-entraining agents were used: Darex-AEA, Vinsol resin, and 7D4 (a new agent developed at the University of California).

Four series of mixes were used with sand contents of 45, 42, 39 and 36 percent of the total weight of air dried sand and coarse aggregate. In each series of mixes the sand grading was varied. Nine different fineness moduli were used ranging from 3.62 to 1.98 (Fig. 1). The maximum size of coarse aggregate was ¾ in., and the aggregate was varied slightly

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†Member American Concrete Institute, Certified Concrete Ltd., Wellington, New Zealand.

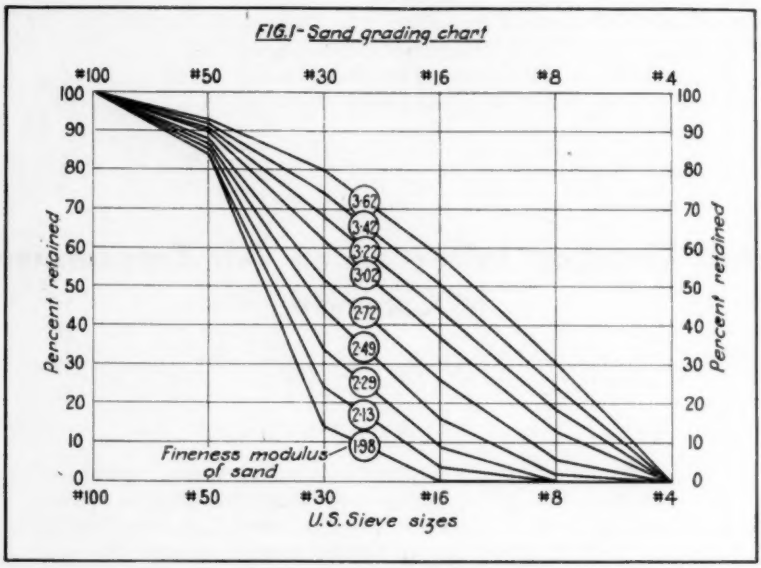


Fig. 1—Sand grading chart

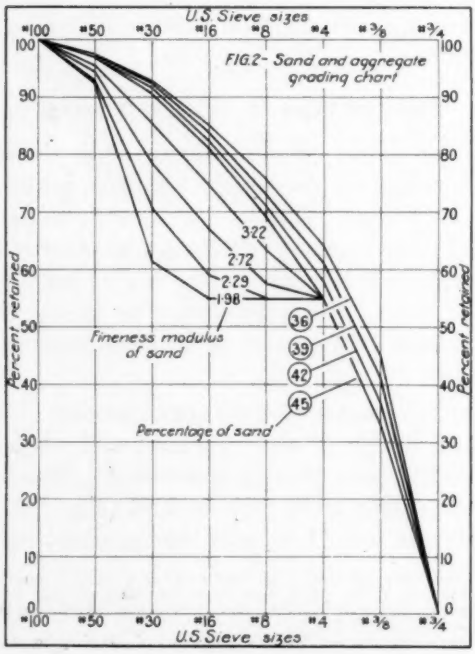


Fig. 2—Sand and aggregate grading chart.

for each series as indicated in the sand and aggregate grading chart (Fig. 2).

The cement used was Type I normal portland cement containing 48 percent C_3S, 26 percent C_2S and 13 percent C_3A. Coarse aggregates were of two sizes ¾–⅜ in. and ⅜–No. 4, of a naturally formed and rounded alluvial gravel. Sand batches were made up from supplies of the following grades:

No. 4	–	No. 8
No. 8	–	No. 16
No. 16	–	No. 30
No. 30	–	No. 50
No. 50	–	No. 100

The graded supplies were obtained by passing standard laboratory supplies of sand over a vibrating screen and repeating the operation until by test it was found that no more than 2 percent of finer grade was present.

Concrete was mixed in a ¾ cu ft electrically powered open type tilting mixer operating at 25 rpm. Before each mix the mixer drum was thoroughly washed and the surfaces left moist. The batch of coarse aggregate and sand was placed in the mixer and mixed for ½ minute before adding the cement. The procedure was as follows:—

Minutes elapsed time*	Sequence of events
0	Cement added
1	Water and agent added
5	Concrete dumped on to steel pan
8	Slump cone removed and slump measured
10	Remolding blows commenced
14	Flow blows commenced

*From time of adding cement to mixers.

TESTS MADE

The percentage of air entrained was measured gravimetrically, and by the use of a pressure meter similar to the Klein-Walker meter described in the ACI JOURNAL, June 1946, *Proceedings* V. 42, p. 657. The values for entrained air here given are those obtained by the pressure method and do not include the percentage of air present in the aggregate. The percentage of air as determined by the pressure method was, in all but two cases, higher than that obtained gravimetrically and owing to the inconsistent results obtained by the latter method it was decided to discard these for the comparisons made in this program.

In each group of tests at the same nominal cement factor the batch weight of cement was kept constant, this weight being calculated by assuming average water-cement ratios and air contents for the whole group. Actual cement factors therefore varied slightly from the nominal

cement factor. Maximum and minimum values of cement factors are given in the following table.

TABLE 1—MAXIMUM AND MINIMUM CEMENT FACTORS

Nominal cement factor, sacks per cu yd	Actual cement factor, sacks per cu yd	Fineness modulus of sand	Proportion of sand, percent	Entrained air, percent	*W/C* by wt.
6	Maximum 6.073	3.22	36	3.05	0.43
	Minimum 5.623	1.98	45	5.98	0.56
4.5	Maximum 4.702	3.42	36	3.25	0.54
	Minimum 4.342	1.98	45	5.10	0.77

For each condition three 3x6-in. cylinders were standard cured and tested in compression while still moist at the age of 28 days. Comparison of compressive strengths obtained with several mixes using varying air-entraining agents is given in Table 2.*

GENERAL CONCLUSIONS

Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand irrespective of the agent used and these results were similar when no agent was used.

For a nominal cement factor of 6 sacks per cu yd and 36 percent sand, the percentage of air entrained reaches a maximum when the fineness modulus of the sand is 2.20 as shown in Fig. 3a. As shown in Fig. 3b, for a nominal cement factor of 4.5 sacks per cu yd and 45 percent sand, the percentage of air entrained reaches a maximum when the fineness modulus of the sand is 2.50, and when 42 percent of sand is used a maximum value of air entrained is reached when the fineness modulus of the sand is 2.20. More data comparing effects achieved with different air-entraining agents are given in Fig. 4.

The quantity of air entrained appears to be a function of the quantity of No. 30–No. 50 sand and this relation is indicated in Fig. 5a and 5b.

Air entrainment increased with the proportion of sand used in a mix as indicated in Fig. 6a and 6b. For a nominal cement factor of 6 sacks per cu yd the increase becomes greater as the fineness modulus of the sand is decreased. For a nominal cement factor of 4.5 sacks per cu yd this increase reaches a maximum when the fineness modulus of the sand is 2.50, and then becomes less for a further decrease in fineness modulus.

*Additional tabular data available from the Institute at cost of reproduction list percent and fineness modulus of sand, percent of No. 30-No. 50 sand, *W/C*, remold and flow test results, air entrainment, unit weight of fresh and hardened concrete, and compressive strength for all of the mixes used in the test program.

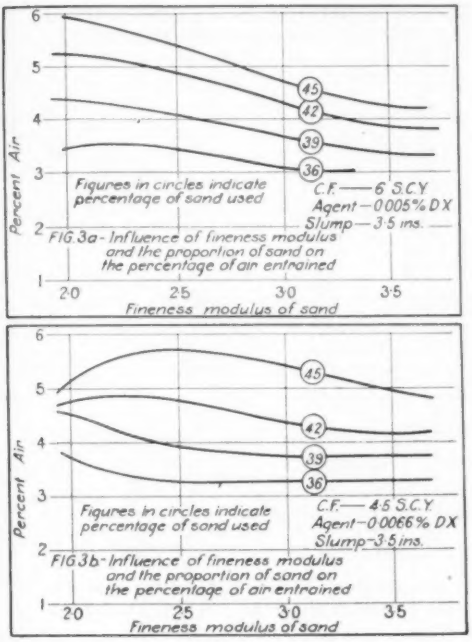


FIG. 3a - Influence of fineness modulus and the proportion of sand on the percentage of air entrained

FIG. 3b - Influence of fineness modulus and the proportion of sand on the percentage of air entrained

Fig. 3—Influence of fineness modulus and the proportion of sand on the percentage of air entrained. a. (top) —cement factor, 6 sacks per cu yd. b. (bottom) — cement factor, 4.5 sacks per cu yd.

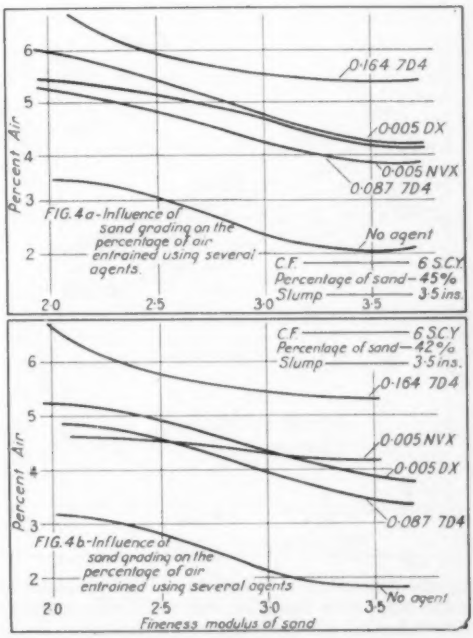


FIG. 4a - Influence of sand grading on the percentage of air entrained using several agents.

FIG. 4b - Influence of sand grading on the percentage of air entrained using several agents

Fig. 4—Influence of sand grading on the percentage of air entrained using several agents. a. (top)—percentage of sand, 45. b. (bottom)—percentage of sand, 42.

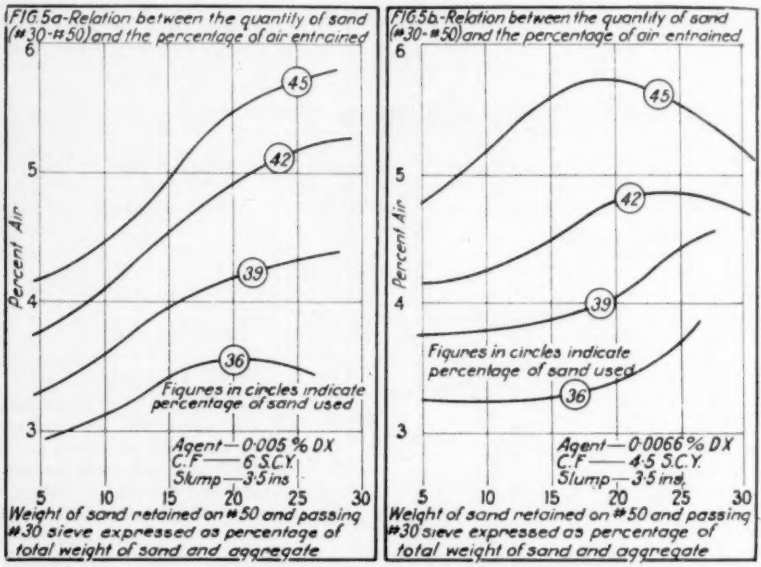


FIG. 5a-Relation between the quantity of sand (#30-#50) and the percentage of air entrained
Percent Air
45
42
39
36
Figures in circles indicate percentage of sand used
Agent — 0·005 % DX
C.F — 6 S.C.Y.
Slump — 3·5 ins
Weight of sand retained on #50 and passing #30 sieve expressed as percentage of total weight of sand and aggregate
FIG. 5b-Relation between the quantity of sand (#30-#50) and the percentage of air entrained
Agent — 0·0066 % DX
C.F — 4·5 S.C.Y.
Slump — 3·5 ins.

Fig. 5—Relation between the quantity of sand (No. 30-No. 50) and the percentage of air entrained. a. (left)—cement factor, 6 sacks per cu yd. b. (right)—cement factor, 4.5 sacks per cu yd.

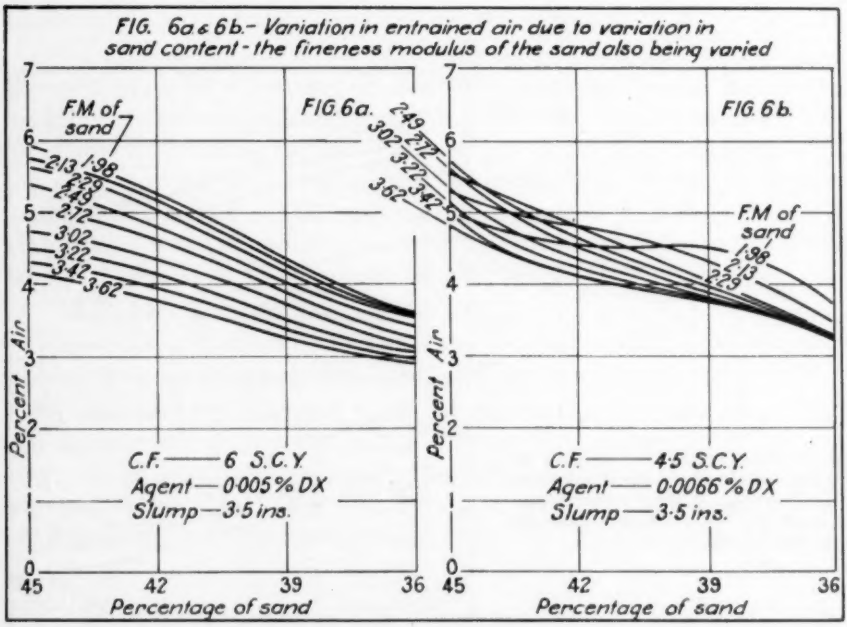


Fig. 6—Variation in entrained air due to variation in sand content—the fineness modulus of the sand also being varied. a. (left)—cement factor, 6 sacks per cu yd. b. (right)—cement factor, 4.5 sacks per cu yd.

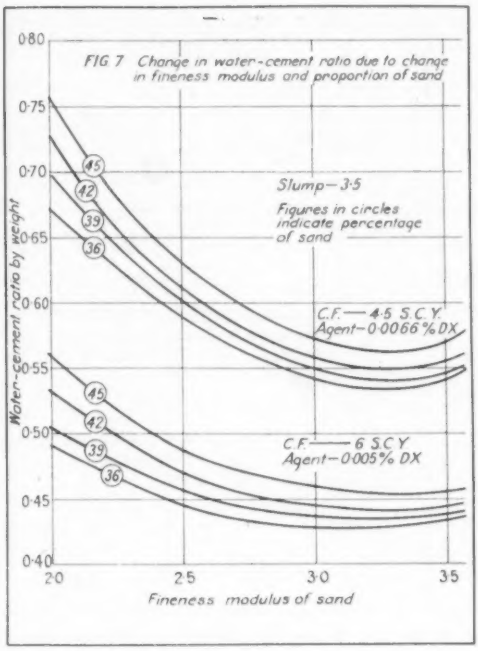


Fig. 7—Change in water-cement ratio due to change in fineness modulus and proportion of sand.

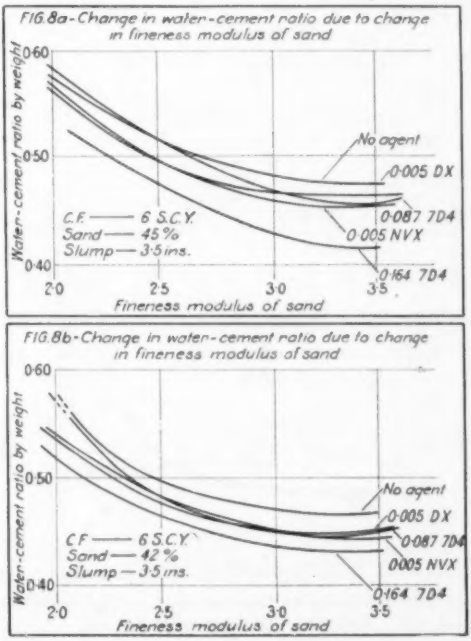
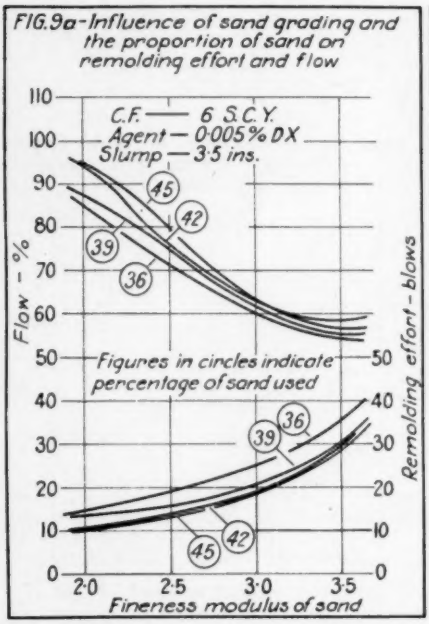


Fig. 8—Change in water-cement ratio due to change in fineness modulus of sand. a. (top)—percent of sand, 45. b. (bottom)—percent of sand, 42.



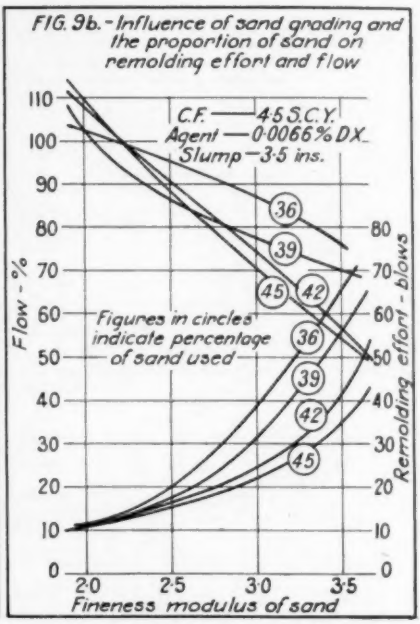
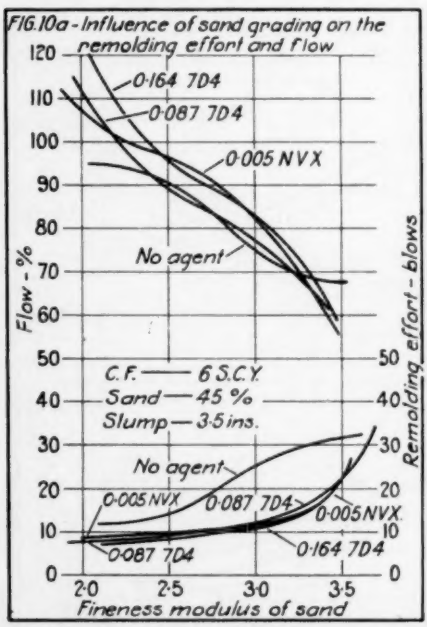


Fig. 9—Influence of sand grading and the proportion of sand on remolding effort and flow. a. (left)—cement factor, 6 sacks per cu yd. b. (right)—cement factor, 4.5 sacks per cu yd.



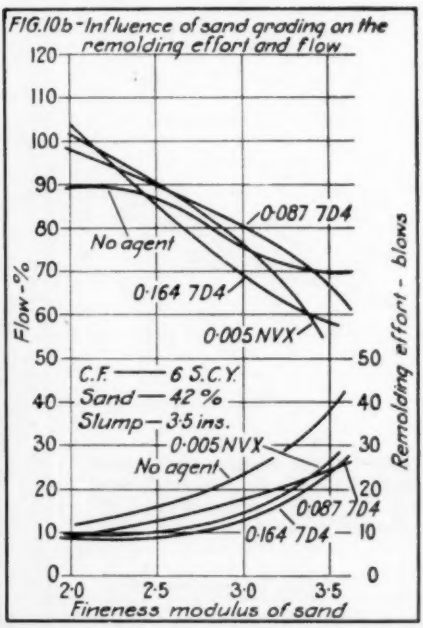


Fig. 10—Influence of sand grading on the remolding effort and flow. a. (left)—percent of sand, 45. b. (right)—percent of sand, 42.

As the proportion of sand in a concrete mix decreases, so does the water-cement ratio decrease. For any proportion of sand between 45 and 36 percent, the water-cement ratio is a minimum when the fineness modulus of the sand is approximately 3.30 (Fig. 7). For fineness moduli of sand less than 3.30, the water-cement ratio increases; also for fineness moduli greater than 3.30 the water-cement ratio definitely increases, though the rate of increase is small. These characteristics are the same for all agents used as indicated in Fig. 8a and 8b.

As the proportion of sand decreases, so do the values for flow (as determined by flow table) decrease, and the values for remolding effort (as determined by Powers' remolding apparatus) increase. As the fineness modulus of the sand decreases, so do the values for flow increase and the values for remolding effort decrease (Fig. 9a and 9b). These characteristics appear to be similar for all agents used (Fig. 10a and 10b). This illustrates the limitations of the slump test as a measure of workability.

Maximum strengths were obtained when the fineness modulus of the sand was approximately 3.20. When the fineness modulus of the sand was less or greater than 3.20 the strengths were lower. The fineness modulus of sand has less effect on strength when the proportion of sand in the mix is low (36 percent) than when the proportion of sand

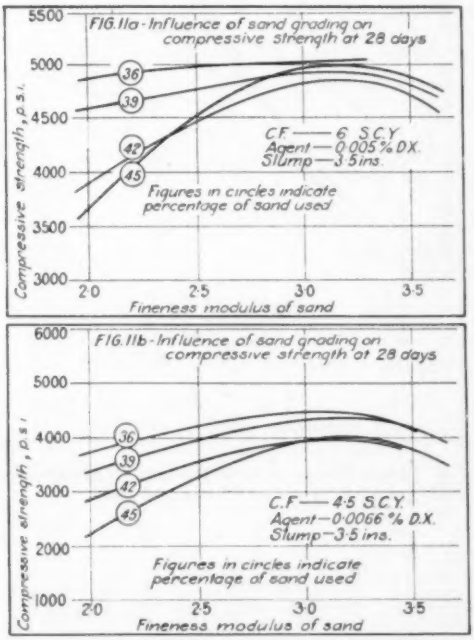


Fig. 11—Influence of sand grading on compressive strength at 28 days. a. (top)—cement factor, 6 sacks per cu yd. b. (bottom)—cement factor, 4.5 sacks per cu yd.

is high (45 percent). These characteristics are illustrated in Fig. 11a and 11b.*

Lower strengths were obtained with Darex and Vinsol resin than were obtained with plain concrete. Higher strengths were obtained with 7D4 than were obtained with plain concrete except when the fineness modulus of the sand was less than approximately 2.70 (Fig. 12a and 12b). In spite of a considerable percentage of air entrained, the use of 7D4 gave higher strengths than could normally be obtained without air entrainment.

While low strengths were obtained for low fineness moduli of sand, because of the higher percentage of air entrained and higher water-cement ratios the concrete was very much more workable, even though the slump always remained constant.

*Complete tabular data from which the charts in this paper were prepared are available from the Institute at cost of reproduction. These tables list percent and fineness modulus of sand, percent of No. 30-No. 50 sand, W/C, remold and flow test results, air entrainment, unit weight of fresh and hardened concrete, and compressive strength for all of the mixes used in the test program.

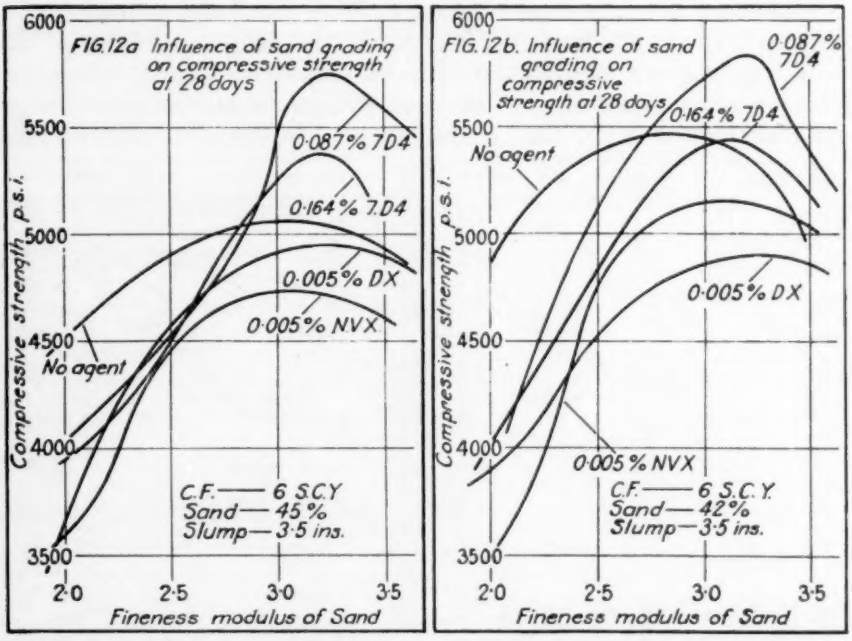


Fig. 12—Influence of sand grading on compressive strength at 28 days. a. (left)—45 percent sand. b. (right)—42 percent sand.

TABLE 2—COMPARISON OF COMPRESSIVE STRENGTHS OBTAINED WITH SEVERAL AGENTS

Fineness modulus of sand	Air-entraining agent	Wt. of agent, percent by wt. of cement	Entrained air, percent	*W/C* by wt.	Compressive strength, psi	
					7 days	28 days
3.42	None	–	1.88	0.46	2960	5100
	DX	0.005	3.90	0.45	2930	4880
	NVX	0.005	4.19	0.44	3970	5090
	7D4	0.087	3.48	0.44	4310	5530
	7D4	0.164	5.33	0.42	3950	5280
2.49	None	–	2.80	0.49	3060	5270
	DX	0.005	4.80	0.47	2700	4490
	NVX	0.005	4.52	0.48	3000	4750
	7D4	0.087	4.56	0.47	3910	5020
	7D4	0.164	5.85	0.46	3730	4750

Slump—3.5 in.
Cement factor—6 sacks per cu yd
Sand—42 percent.

Here again a study of air entrainment—two papers in this issue (see also p. 205). It is clear that the subject is not exhausted. Air in the concrete *is* a means to durability, partially upsetting early ideas of density, as some performance records show. But air-entrainment is not in itself a cure. The question is "Air plus what other factors?". This is likely to be debated at ACI's 45th annual convention, New York in February in an entire session on durability problems in concrete paving.

Influence of Size Grading of Sand on Air Entrainment*

By E. W. SCRIPTURE, Jr.,† F. B. HORNIBROOK‡ and D. E. BRYANT§

SYNOPSIS

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28–48 mesh size sand rather than the 48–100 mesh size.

INTRODUCTION

The entrainment of air in concrete to increase durability has become widely accepted. This entrainment of air is secured either by intergrinding certain foaming agents with the cement in the process of manufacture or by the addition at the mixer of small amounts of certain admixtures which cause air entrainment. It is also generally accepted that the percentage of air which should be entrained in the concrete should lie between 3 and 5 percent, or between 3 and 6 percent, of the volume of the concrete. Below the minimum the full effects of increased durability are not secured and above the maximum serious losses in strength may result without any appreciable gain in durability. Control

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of the percentage of air entrained, therefore, becomes a matter of some importance.

There are a large number of factors which affect, or are thought to affect, the amount of air entrained. These include the nature of the air-entraining agent itself, the proportion of air-entraining agent, the manner in which it is used, mix design, sand-total aggregate ratio, mixing time, type of mixer, temperature and inherent characteristics of the cement. Another factor which it is commonly believed has an important influence on air entrainment is the size grading of the sand used in the concrete mix.

The idea that air entrainment is markedly affected by the size grading of the sand appears reasonable but the only published work on this specific subject seems to be a limited investigation by Henry L. Kennedy*[1,2] and a briefly reported study by Walker and Bloem.[3] Kennedy says:

> In this connection our investigation of the sand constituent showed that air entraining agents were not effective on size particles larger than retained on 14 mesh. Hence a study of the various particle sizes of sand finer than No. 14 in the presence of air entraining agents, was conducted. The average relative amount of air entrained by the various sand fractions using identical mixing procedure, were as follows:

Sieve size	Percent air
14–28	15–20
28–48	30–35
48–100	45–50
Finer than 100	0–1

> It is obvious that radical changes in sand gradation could alter materially the total air entrained in a concrete. It is also obvious that air entraining agents will show to greater advantage in lean mixes than in rich mixes.

Kennedy's work was carried out on mixes consisting of sand, water and air-entraining agent only, that is, without cement or coarse aggregate. There is some question how far results on this type of mix are applicable to concrete or mortar mixes. In order to secure more conclusive data on this subject this investigation was undertaken.

TEST PROGRAM

The general program consisted of making mixes and determining air contents, with no air-entraining agent and with three different air-entraining agents, of:

*See references at end of text. Also published in this JOURNAL, p. 205 is another paper on the same topic: "Sand Grading Influence on Air Entrainment in Concrete," by M. A. Craven.

1. Mixes of sand and water only
2. Mortar mixes, 1:2 and 1:4
3. Concrete mixes.

In the mixes with sand and water only and the 1:2 mortar and one series of 1:4 mortar mixes, sands were used consisting solely of one sieve size. In a second series of 1:4 mortar mixes and the concrete mixes, sand of a normal grading was used within each case, one size greatly exaggerated by the addition of extra sand of that particular sieve size. A recombined sand was also prepared and included in some of the test series having approximately what might be called an ideal grading. Screen analyses of these recombined sands are given in Table 1.

The sand used was a lake sand, specific gravity 2.65, screened into seven fractions. The coarse aggregate used in the concrete mixes was a 1-in. crushed limestone, specific gravity 2.65, having a size grading:

Retained on 1-in. sieve	7 percent
Retained on ¾-in. sieve	26 percent
Retained on ½-in. sieve	48 percent
Retained on ⅜-in. sieve	15 percent
Retained on No. 4 mesh	4 percent
Passing No. 4 mesh	0 percent

The cement used was standard portland cement, Type I, having a surface area of 1550 sq cm per g. The air-entraining agents were:

A. Neutralized gasoline insoluble pine tree resin
B. Sulfonated hydrocarbon
C. Cement dispersing agent

For the sand and water mixes and the mortar mixes the mixing was done and the air contents were determined in accordance with standard A.S.T.M. method C 185-46T except, of course, that the actual mixes differed from that specified in this method in the cases of the sand-water mixes and the 1:2 mortar. The concrete mixes were made in batches of approximately 2 cu ft, mixed 3 minutes in a 2½ cu ft concrete mixer, and the air contents were determined immediately after mixing with a Klein-Walker air meter. The water contents of all mixes and the slumps of the concrete mixes are given in Tables 2 and 3.

TEST RESULTS

Sand-water mixes

Considering the mixes made with sand and water only, that is, without cement, the relations are illustrated in Fig. 1, on which are also shown the average values found by Kennedy for this type of mix. It will be seen that with either air-entraining agent A or air-entraining agent C the amount of air entrained increases, as the size of the sand decreases, to a

TABLE 1—SCREEN ANALYSES OF RECOMBINED SANDS

Mix	Predominating size—between screen No.	Sand grading, percent retained on sieve No.							Fineness modulus
		4	8	14	28	48	100	−100	
1:4 Mortar mixes	4–8	1.5	30.0	18.0	19.0	18.0	11.0	2.5	3.35
	8–14	1.0	10.0	40.0	18.5	17.5	10.5	2.5	3.17
	14–28	1.0	10.0	18.0	40.0	18.0	10.5	2.5	2.94
	28–48	1.0	10.0	17.5	18.5	40.0	10.5	2.5	2.72
	48–100	2.0	10.5	18.0	19.0	18.0	30.0	2.5	2.59
	Through 100	2.0	11.0	18.5	19.0	18.5	11.0	20.0	2.46
	Ideal grading	2.5	12.5	22.0	23.0	22.0	14.0	4.0	2.92
Concrete mixes—Admixture A	4–8	3	36	17	16	17	9	2	3.57
	8–18	3	14	39	16	17	9	2	3.35
	14–28	3	14	17	38	17	9	2	3.13
	28–48	3	14	17	17	38	9	2	2.92
	48–100	3	14	17	17	17	31	2	2.69
	Through 100	3	14	17	17	16	9	25	2.45
	Ideal grading	4	18	22	20	21	12	3	3.15
Concrete mixes—Admixture C	4–8	3	26	16	19	19	14	3	3.21
	8–14	3	10	32	19	19	14	3	3.05
	14–28	3	10	16	35	19	14	3	2.89
	28–48	3	10	16	19	35	14	3	2.73
	48–100	3	10	16	19	19	30	3	2.57
	Ideal grading	3	12	18	23	23	17	4	2.82
Concrete mixes—Normal vs. screened sand	Normal	2	11	14	18	37	17	1	
	Screened	3	17	19	22	36	3	0	

TABLE 2—WATER CONTENT OF SAND AND MORTAR MIXES

Sand-water mixes

Addition	Percent water, by wt. of sand, when sand size is					
	4–8	8–14	14–28	28–48	48–100	Through 100
None	17.5	18.3	19.2	20.8	23.4	25.0
Admixture A	16.6	17.5	18.3	19.2	20.8	22.3
Admixture C	16.6	17.5	18.3	19.2	20.8	22.3

Mortar mixes—sands all of one size

Addition	Water-cement ratio by wt., when sand size is					
	4–8	8–14	14–28	28–48	48–100	Through 100
1:4 Mortar						
None	0.633	0.667	0.867	1.033	1.217	1.333
Admixture A	0.583	0.633	0.833	0.967	1.167	1.283
Admixture B	0.583	0.633	0.833	0.967	1.167	1.283
Admixture C	0.583	0.633	0.800	0.933	1.133	1.250
1:2 Mortar						
None	0.372	0.429	0.485	0.543	0.600	
Admixture A	0.343	0.372	0.429	0.485	0.543	
Admixture C	0.343	0.372	0.429	0.485	0.543	

1:4 Mortar mixes—sands having a predominating size

Addition	Water-cement ratio by wt., when predominating sand size is						
	Ideal	4–8	8–14	14–28	28–48	48–100	Through 100
None	0.700	0.633	0.667	0.700	0.733	0.733	0.767
Admixture A	0.633	0.617	0.633	0.667	0.700	0.733	0.733
Admixture B	0.633	0.617	0.633	0.667	0.700	0.733	0.733
Admixture C	0.633	0.600	0.633	0.650	0.700	0.733	0.733

TABLE 3—WATER CONTENT AND SLUMP, CONCRETE MIXES

Sieve Size	Admixture A (0.015 percent)		Admixture A (0.02 percent)		Design method A				Design Method B			
					No addition		Admixture C		No addition		Admixture C	
	W/C	Slump	*W/C*	Slump	*W/C*	slump	*W/C*	slump	*W/C*	slump	*W/C*	slump
Ideal	5.62	2¾	5.47	3¼	6.46	4	5.60	3½	6.46	4	5.60	3½
4– 8	—	—	5.59	2¾	6.23	3½	5.61	4	—	—	—	—
8– 14	—	—	5.58	2¾	6.31	4	5.56	3½	6.16	3½	6.06	3¼
14– 28	—	—	5.52	2¾	6.29	3¼	5.56	3½	—	—	—	—
28– 48	—	—	5.48	3	6.46	3½	5.71	3½	—	—	—	—
48–100	—	—	5.48	2¾	6.46	3	5.90	3¾	6.20	3¾	5.77	4
Thr. 100	—	—	—	—	6.76	3½	6.06	3¼	—	—	—	—

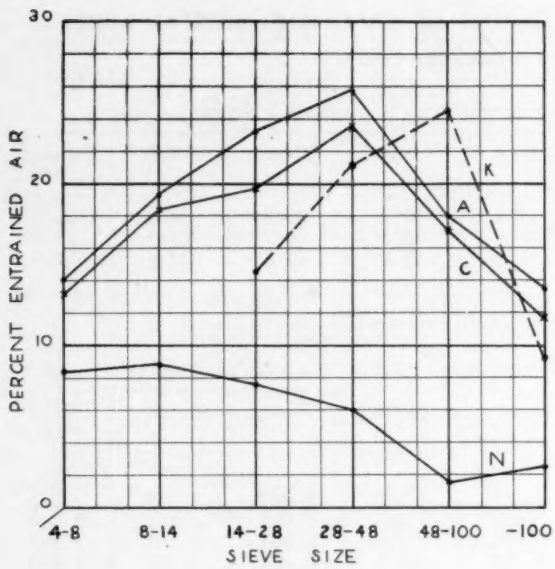

Fig. 1—Sand-water mixes
A—Air-entraining agent A
C—Air-entraining agent C
N—No air-entraining agent
K—Kennedy average values, A.S.T.M. paper

maximum at the No. 28-48 mesh sieve. Thereafter, the amount of air entrained decreases rapidly. This is in agreement with Kennedy's findings to the extent that there is a sieve size at which maximum air entrainment is secured, but in the case of Kennedy this was 48–100 mesh instead of 28–48 mesh. That the 28–48 mesh size has the greatest effect on air entrainment is in agreement with the findings of Walker and Bloem. Both this investigation and Kennedy's show that air entrainment is very low with the finest material (minus 100 mesh). Contrary to Kennedy's finding, appreciable air entrainment is secured with the coarser sizes, even including the 8–14 mesh size. The general shapes of the curve from this work and that from Kennedy's investigation are similar, and the differences in their positions might be explained by the fact that two sands probably of totally different nature were used.

With no air-entraining agent some air is entrained but there is no very well defined maximum. It is doubtful that there is any significant difference in the amount of air entrained with any of the sizes from 4–8 mesh to 28–48 mesh. The two finer sizes definitely entrain less air.

Mortar mixes

With the 1:4 mortars, using sands of a single sieve size, for which the results are shown graphically in Fig. 2a, the data are generally similar to those secured with the sand-water mixes. The amount of air entrained in all cases is, however, on a lower level. With all three air-entraining agents and also in the mixes without air-entraining agents, a maximum amount of air is entrained by using the 14–28 mesh size. This indicates

a shift from the maximum of 28–48 mesh with sand alone. Again all data are in agreement that the two finer sizes entrain very much less air. The shapes of the curves for the three air-entraining agents, A, B and C, are very similar. Their relative positions have no significance, as it is obviously possible to move any one of them up or down by changing the proportion of air entraining agent used.

In Fig. 2b the results of the 1:4 mortar tests, using graded sands with one sieve size predominating rather than sands of a single sieve size, are shown graphically. In general, the shapes of the curves are similar to those secured with the single sieve size sands but the differences are less accentuated and the maxima are less well defined. Maximum air entrainment might be said to occur with either the 14–28 or the 28–48 mesh size predominating. All the data are again in agreement that the finer fractions show decreased air entrainment.

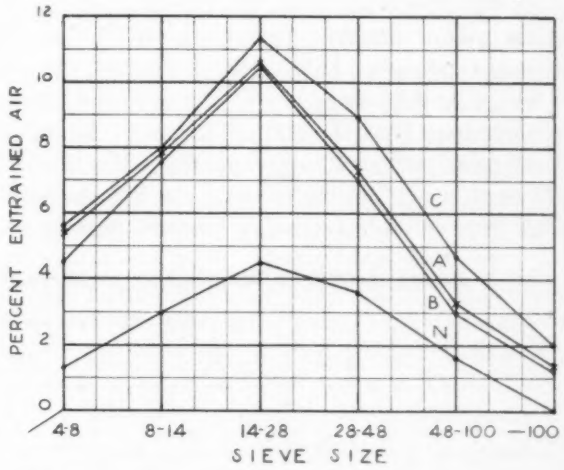

Fig. 2a — 1:4 mortars, sands of a single sieve size

A—Air-entraining agent A
B—Air-entraining agent B
C—Air-entraining agent C
N—No air-entraining agent

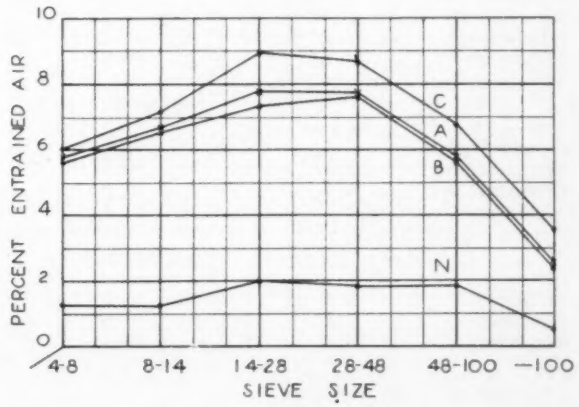

Fig. 2b — 1:4 mortars, sands having one predominating sieve size

A—Air-entraining agent A
B—Air-entraining agent B
C—Air-entraining agent C
N—No air-entraining agent

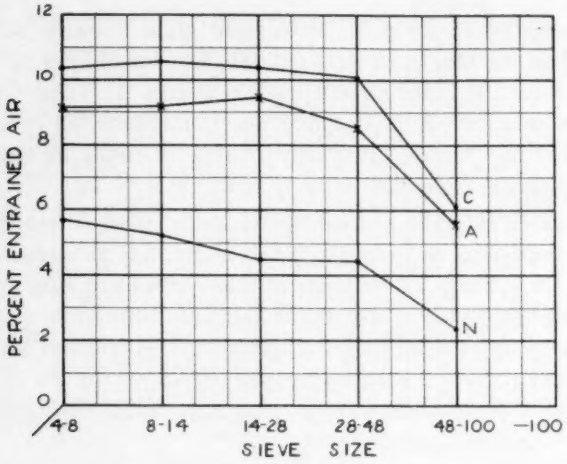

Fig. 3—1:2 mortars, sands of a single sieve size
A—Air-entraining agent A
C—Air-entraining agent C
N—No air-entraining agent

In Fig. 3 the data from the tests on 1:2 mortars using sands of a single sieve size are plotted. The amount of air entrained in this type of mortar does not appear to be greatly affected by the sieve size between the sizes 4–8 mesh and 28–48 mesh. The amount of air entrained falls off sharply, however, with the 48–100 mesh size. The 1:2 mortars show no well-defined peaks, which is very much in contrast to the 1:4 mortars. That the air contents of the 1:2 mortars are, for the most part, as high as or higher than those of the 1:4 mortars, is surprising but may be without significance

TABLE 4—AIR ENTRAINED BY SANDS OF ALL ONE SIZE

	Percent of entrained air for each sieve size					
	4–8	8–14	14–28	28–48	48–100	–100
Sand only						
Kennedy in ref. 1	—	—	15–20	30–35	45–50	0–1
Kennedy in ref. 2	—	—	13.1–15.7	20–22.3	23.2–26	7.9–10.3
No addition	8.3*	8.9*	7.6*	6.0*	1.4*	2.6*
Admixture A	14.2*	19.4*	23.2*	25.9*	18.1*	13.4*
Admixture C	13.2*	18.5*	19.8*	23.8*	17.1*	11.7*
1:4 mortar						
No addition	1.3	3.0	4.5	3.6	1.6	0.0
Admixture A	5.5	7.7	10.6	7.2	3.2	1.3
Admixture B	4.5	7.7	10.6	7.0	3.0	1.2
Admixture C	5.5	8.0	11.4	9.0	4.6	2.0
1:2 mortar						
No addition	5.6	5.2	4.5	4.5	2.4	—
Admixture A	9.1	9.2	9.5	8.6	5.5	—
Admixture C	10.3	10.5	10.4	10.1	6.1	—

*Average of two determinations.

TABLE 5—AIR ENTRAINED BY SANDS HAVING A PREDOMINATING SIZE —1:4 MORTAR

	Percent air entrained for sand of predominating seive size						
	Ideal	4–8	8–14	14–28	28–48	48–100	–100
No addition	2.1	1.2	1.2	2.0	1.8	1.8	0.5
Admixture A	7.2	5.7	6.6	7.7	7.7	5.8	2.4
Admixture B	7.0	5.6	6.5	7.3	7.6	5.7	2.4
Admixture C	7.5	6.0	7.1	8.9	8.6	6.8	3.5
Fineness modulus	2.92	3.35	3.17	2.94	2.72	2.59	2.46

as the two series were rather widely separated in time and there may have been some change in conditions which produced this effect. Data for all the air determinations on the mixes of sand only and on the 1:4 and 1:2 mortars are given in Tables 4 and 5.

Concrete mixes

Tests on concrete mixes were made in the first place with two mix designs only, one for the series of mixes with no air-entraining agent, and another in which the sand-total aggregate ratio was decreased to allow for entrained air for the mixes using air-entraining agent. This is somewhat unusual, in that no allowance in the mix design is made for the difference in fineness moduli of the sands used, but was done in order to eliminate the influence of mix design on air entrainment. Data from the mixes with no air-entraining agent, with air-entraining agent A and with air-entraining agent C are shown in Fig. 4. It is evident that there are no significant differences in the amounts of air entrained with no air-entraining agent and with admixture C, regardless of the predominating sieve size of the sand. Such small differences as occur are well within

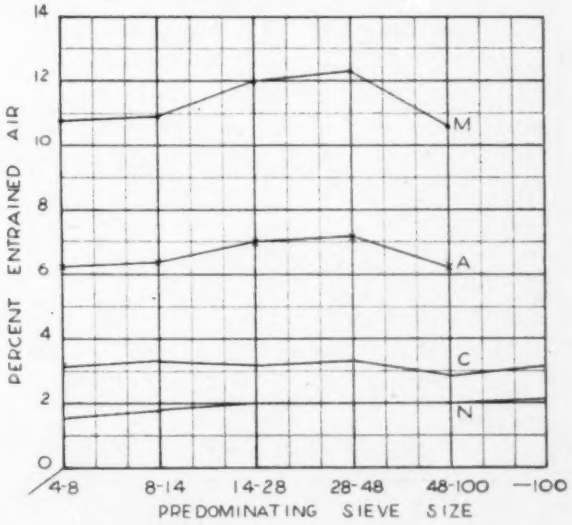

Fig. 4—Concrete mixes
A—Air entraining agent A
C—Air-entraining agent C
N—No air-entraining agent
M—Air-entraining agent A calculated as percent of mortar

TABLE 6—CONCRETE MIXES—AIR ENTRAINED BY SANDS HAVING A PREDOMINATING SIZE

Mix design: Cement—22 lb
Sand—56¾ lb
Stone—78½ lb

Sieve size	Fineness modulus	Entrained air, percent	
		Admixture A (0.015 percent)	Admixture A (0.02 percent)
Ideal	2.82	5.2	7.0
4– 8	3.21	—	6.2
8– 14	3.05	—	6.3
14– 28	2.89	—	7.0
28– 48	2.73	—	7.1
48–100	2.57	—	6.2

Mix designs	Method A		Method B			
	(F.M. 3.15)		(F.M. 3.57)		(F.M. 2.69)	
	No add'n	Admix. C	No add'n	Admix. C	No add'n	Admix. C
Cement, lb	22	22	22	22	22	22
Sand, lb	59½	56¾	67¾	64¾	53	50¼
Stone, lb	80¾	83¾	72¼	75¼	87¾	90½
Sand/total agg.	.425	.405	.485	.465	.375	.355

Predominating size	Air entrained, percent			
	Design method A		Design method B	
	No Add'n	Admixture C	No Add'n	Admixture C
Ideal	2.0	3.1	2.0	3.1
4– 8	1.6	3.1	—	—
8– 14	1.8	3.3	1.7	2.7
14– 28	2.0	3.2	—	—
28– 48	2.0	3.3	—	—
48–100	2.0	2.9	1.8	2.8
Thr. 100	2.2	3.2	—	—

TABLE 7

	Percent entrained air		
	No addition	Admixture A	Admixture C
Nominal slump—4½ in.			
Normal sand	1.2	3.6	2.0
Screened sand	1.3	4.0	3.2
Nominal slump—7½ in.			
Normal sand	0.8	3.1	1.1
Screened sand	0.8	3.4	2.1

the limits of accuracy of the method. When the first two sets of tests were made without air-entraining agent and with air entraining agent C, it was thought that it might be possible that the comparatively low level of air entrainment in these mixes was such as to obscure any significant differences. The series with air-entraining agent A was therefore made with an increased amount of air-entraining agent to secure higher air contents, about at the extreme limit of the range usually considered desirable. With this air-entraining agent there does appear to be a peak for the 14–28 and 28–48 mesh sizes but it is not large. The total spread from highest to lowest air contents is less than 1 percent of entrained air.

It was thought possible that if the concrete mixes were designed, taking into account the fineness modulus of the sand where one size predominated, some further light would be shed on this subject. The results secured with three different designs for widely separated fineness moduli —2.69, 3.15, and 3.57—are given in Table 6. It is evident that there are no significant differences with the different sizes of sand nor between the mixes designed in this manner and the single mix designed without regard to fineness modulus.

Since the conclusions reached on the basis of the data developed in this investigation were contrary to published information and general opinion with respect to the effect of the 48–100 mesh fraction of the sand, a further series of concrete mixes was made to compare directly air entrainment in concrete with a sand containing a normal amount of this fraction and the same sand with practically all material through 48 mesh removed. The size gradings of these two sands are included in Table 1. Twelve concrete mixes with a nominal cement factor of 5 sacks per cu yd were made, six at a nominal slump of 4½ in and six at 7½ in. In each set of six, three mixes were made with the normal sand and three with the same sand screened to remove minus 48 mesh material. In each set of three, one mix was made without air-entraining agent and the other two with air-entraining agents A and C. The results of the air determinations are shown in Table 7:

It may be questioned that any of the differences in air content between the normal and the screened sands are significant; only in the case of admixture C is this difference over 0.4 percent. Insofar as there are differences, however, they are in the direction of higher air content with the sand having the minus 48 mesh fraction removed. This confirms the conclusion drawn from the other data that the 48–100 mesh fraction of the sand does not increase the amount of air entrained in concrete mixes.

CONCLUSIONS

The conclusions reached from the results of this investigation are:

1. In mixes employing sand only without cement, the amount of air entrained is markedly affected by the size grading of the sand. The amount of air entrained

rises with decreasing size of the sand to a maximum for the size 28-48 mesh, and thereafter decreases rapidly. This observation is generally in accord with the work of Kennedy, but differs in the point at which the maximum occurs.

2. In mortar mixes effects are observed similar to those obtained with mixes using sand and water without cement, but the differences with size grading are less marked. It also appears that the point at which maximum air entrainment is secured moves toward the coarser sizes. It is fairly definitely shown that the sizes 48-100 mesh and minus 100 mesh contribute very little to air entrainment. The variations in air entrainment as affected by size grading become less marked as the mortar becomes richer.

3. In concrete mixes the influence of the size grading of the sand on air entrainment is further suppressed. In normal concrete mixes size grading of the sand has no significant effect on the amount of air entrained with air-entraining agent C but there is some evidence of a maximum with agents of the type of admixture A.

The general impression that the amount of air entrained in concrete mixes is markedly affected by the size grading of the sand used in the mix, is probably derived from the results reported by Kennedy on sand-water mixes, which are generally confirmed by this investigation. In mortars size grading has an influence on air entrained, more marked with the leaner mortars. In concrete the effect is suppressed to a very considerable extent so that the variations in air content with size grading of the sand are quite small.

While it is indicated that the effect of size grading of the sand on air entrainment in concrete mixes is relatively small, it is still very probable that other factors related to the sand may be of importance. The ratio of sand to total aggregate in the mix is certainly significant, although this is not borne out by the few data reported in the series where the sand-total aggregate ratio was varied to allow for change in fineness modulus. It is also suggested that the particle shape of the sand grains themselves may play a part. Another factor is the consistency of the mix. In this investigation most of the mixes were of moderate slump and the air determinations were made immediately after mixing. With more fluid mixes it is possible that the size grading of the sand would have more effect on the final amount of entrained air, not so much because the fine sizes would entrain more air in the first instance but because they would contribute to the cohesiveness of the mix which would, in turn, tend to allow less of the entrained air to escape subsequently.

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Use of Prepacked Aggregate Concrete in Major Dam Construction*

By PAUL BAUMANN†

SYNOPSIS

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

INTRODUCTION

The expansion of mass concrete due to the heat of hydration and the subsequent contraction due to cooling and dry shrinkage have been recognized as important factors affecting the safety and permanence of structures, particularly major dams. Attempts to overcome these effects through the use of low heat cement, low water-cement ratio and internal cooling systems have met with little success, primarily because of the relatively high cement content required for minimum strength of conventional concrete.

The restoration of Barker Dam as described in the paper‡ by Raymond E. Davis, E. Clinton Jansen and W. T. Neelands stimulated the use of prepacked aggregate concrete in combination with precast form slabs, in the construction of dams. In studying the problems involved it became clear, however, that instead of vertical intrusion pipes with intermittent slots as used at Barker Dam, horizontal pipes of similar size

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‡ACI JOURNAL, April 1948, *Proc.* V. 44, p. 633.

with continuous slots along the bottom and accessible from the downstream side would have distinct advantagse. They would:

1. Simultaneously serve as form ties.
2. Lend themselves readily to pumping of pre-cooling water into the coarse aggregate and post-cooling of the concrete.
3. Make it possible to perform intrusion from the downstream face without interference with the placing of the aggregate.
4. Serve to drain the concrete mass and thereby prevent uplift.

PREPACKED AGGREGATE CONSTRUCTION OF A MAJOR GRAVITY DAM

The Prepakt method was described by Raymond E. Davis and the writer in a proposal for the construction of a major gravity dam in the Swiss Alps in the summer of 1947, in connection with the firm bid of the Walter J. Heller Company, contractors in Bern, Switzerland. The essentials of this proposal were as follows:

After the foundation excavation, grouting and cleanup had been completed, stepped footings upon which to erect the slabs along the upstream and downstream contacts of the faces of the dam were to be constructed of conventional concrete having properties of strength and frost resistance equal to those of the slabs.

Starting at the base of the dam, the graded coarse aggregate ranging in size from about 10mm (3/8 in.) to 300mm (12 in.) was to be placed in horizontal layers over the full horizontal section. At the same time horizontal pipes about 2 in. in diameter with a continuous slot along the bottom, (Fig. 1) spaced about 2m (6.56 ft.) both vertically and horizontally, were to be placed at right angles to the axis of the dam; the precast slabs, which were to act as a form for the aggregate fill, were to be erected and connected to the pipes by means of field welding so as to be in exact alignment with the faces. Fig. 1 shows details of form slab construction and connections with the slotted pipes.

When the assemblage of slabs and aggregate fill had reached the limiting elevation as judged by the two criteria of mass stability and form pressure* the intrusion of mortar (grout) into the aggregate mass was to be commenced and to be continued without interruption until the grout surface had overtaken the aggregate surface. The placing of coarse aggregate would be continued during the intrusion operation, the rise of the latter being more rapid than the rise of the former. This operation which was to transform the mass of aggregate into concrete was to comprise several distinct steps.

*The criterion of mass stability is satisfied if a triangular stable rockfill section with 1½:1 slope can be inscribed in the loose or ungrouted aggregate section so as to have the base of the triangle coincide with the base of the dam (foundation) or with the level of the grouted (intruded) aggregate and the apex of the triangle at the top of the loose aggregate. The form pressure criterion is satisfied if the pressure on the forms and the tension in the pipe ties, which is measured by means of electric strain meters attached thereto, produces stresses which are within safe limits.

Pre-cooling the aggregate

Pre-cooling the aggregate was to be accomplished by inundation with the cold water of the stream, circulating through the pipe system. In this manner the temperature of the aggregate mass prior to the intrusion with mortar would be substantially the same as that of the water supply or approximately 5 C (41 F).

Volumetric determination of void space

The void space of the aggregate mass was to be determined by measuring the volume of water filling the voids. In this manner an exact check

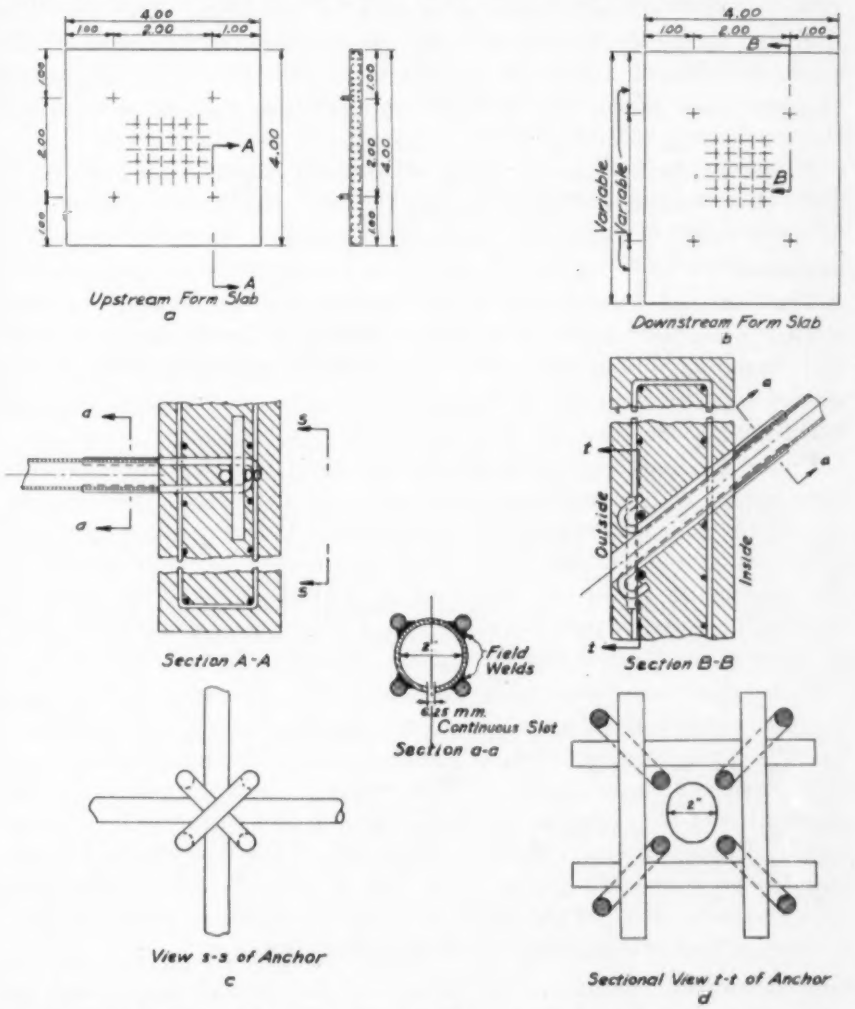

Fig. 1—Details of form slab construction. Parts c and d indicate the method of installing and attaching the slotted pipes used for grouting.

on the volume of mortar required to completely fill the voids would be obtained.

Intrusion of aggregate mass

Intrusion of the aggregate mass was to be done by pumping mortar through inserts in the slotted pipes, beginning at the lowest point, displacing the water with which the voids are filled, in such manner as to maintain a substantially horizontal grout level, which at all points would be in a plastic, flowable condition. The mortar for this work would be composed of approximately 1 part portland cement; 1 part Alfesil, a very finely divided siliceous material capable of combining with the lime liberated during the hydration of the cement; three to four parts of fine sand; an intrusion aid in the amount of about 1 percent of the weight of cement and alfesil; and sufficient water to produce a mixture having the consistency of thick cream.

Assuming the void space of the well graded aggregate fill varying in size between 10 and 300 mm to be 30 percent of the total volume, the cement content of such concrete would be less than two sacks per cu yd.

Post-cooling

The intruded mass would be post-cooled first by washing out the slotted pipes, prior to the time of final setting of the mortar with which they would have been filled, then by circulating cold water through each slotted pipe by inserting therein a smaller pipe connected to the cold water supply system.

Under the foregoing conditions it was calculated that the maximum temperature of the hardening concrete would not exceed the temperature of the cold water supply by more than 8 C and the average maximum temperature of the mass would not exceed by more than 6 C the temperature of the cold water supply. For example, if the temperature of the water supply were 5 C (41 F) the average maximum temperature of the Prepakt mass would not be greater than 11 C (51.8 F).

Under these conditions it was considered that dilatation joints would be unnecessary for the main dam over a length of about 200 m (656 ft) for which the average internal temperature of the mass would vary little from the mean annual. This proposal therefore called for the elimination of the joints as well as the slots in line therewith for a crest length of about 200 m. Hence this portion of the dam would be constructed as a monolith. The wing dam on the left side of the main dam and wing dike on the right side thereof would, however, be constructed with joints as called for in the official drawings.

Uplift elimination

The joints between slabs would be intruded with mortar as the slabs are erected, in a manner similar to that outlined in the paper on Barker

Dam.* Thus there was to be created at the upstream face a membrane relatively impervious as compared with the prepacked mass with its much lower cement content and higher water-cement ratio. If after post-cooling the slotted pipes were allowed to remain open they would serve very efficiently as drains for the removal of any water which might find its way into the mass and would therefore serve to eliminate the possibility of uplift of any consequence. All other conditions remaining unchanged, the elimination of uplift in this manner would permit a reduction in the cross section of the dam which in all probability would more than compensate for the added 32,500 cu yd of concrete made necessary by elimination of the slots at joints.

Thermal shock

It was of course recognized that the concrete at and near exposed faces would be subject to significant temperature changes ordinarily sufficient in magnitude to produce fine surface cracks during cold weather, which would close when the temperature again returned to normal. Under this plan the surface of the mass prepacked concrete would be protected against this thermal shock by the precast reinforced concrete slabs approximately 25 cm (10 in.) thick and 4 by 4 m (13 by 13 ft) in area. In absorbing this thermal shock each joint between the slabs would act as a contraction joint.

Intrusion operations

The coarse aggregate was to be placed at the rate of approximately 195 cu yd per hour or 1950 cu yd for the 10-hour working day.

As previously stated, prior to the injection of intrusion mortar, the aggregate would be inundated with circulating cold water from the stream to a height of perhaps 2 m (6.56 ft) above the foundation rock or level of hardened concrete placed at some earlier date. When the temperature of the coarse aggregate had declined substantially to that of the cold water supply, the operation of intrusion would be started at the lowest point in the forms.

This was to be accomplished by inserting for the full length of each slotted pipe a flexible hose and by gradually withdrawing the insert as the mortar was injected, all in such manner as to maintain the rising surface of the mortar on a substantially horizontal level over the full length and width of the area being intruded. The rising intrusion mortar would displace the cooling water in the aggregate mass in the same manner as the cooling water had displaced the air. The hose in each slotted pipe would be inserted and withdrawn as frequently as necessary to prevent setting of the intrusion material within the pipe and within the channels through which it must move through the aggre-

*"Restoration of Barker Dam," ACI JOURNAL, April 1948, *Proceedings* V. 44, p. 633.

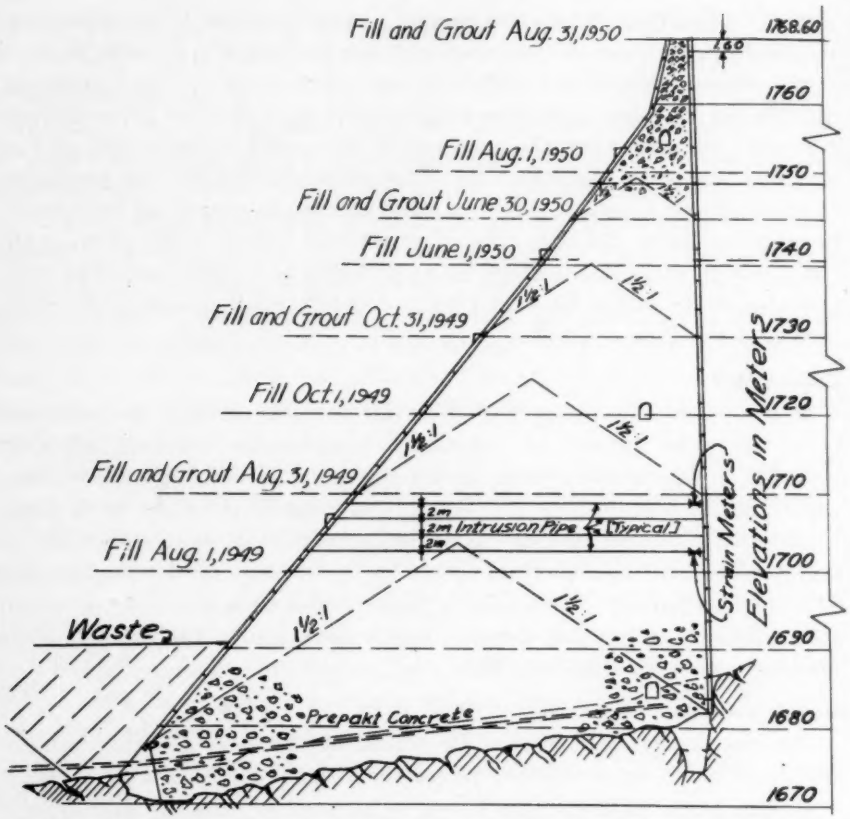

Fig. 2—Maximum section of dam showing proposed schedule of grouting operations

gate mass. In this manner the level of the cooling water and level of the intrusion mortar would be made to rise simultaneously until the level of the aggregate fill were reached. There would be no interruption in the placing of aggregate during the intrusion operation.

It was contemplated that the intrusion operation would be carried out in four steps (see Fig. 2), two during 1949 and two during 1950, each of roughly 30 days duration, when the level of the intrusion mortar would be made to rise at the average rate of about 1 m (3.28 ft.) per day. Once the operation of grouting had been started it would be continued without substantial interruption until completed, that is, until the grout level had risen to the top of the aggregate fill.

Post-cooling as previously outlined was to be conducted over a period of not less than 28 days or until such time as the average temperature of the prepacked concrete was not more than 3 C above that of the cold water supply. In this manner the average temperature of the mass would be

close to the mean annual temperature at the dam site within a period of from one to two months after the initial setting of the cement.

CONSTRUCTION SCHEDULE

The construction schedule (Fig. 2) provided for the work to be started in May 1948, and to complete the entire foundation excavation and foundation grouting and part of the stepped footings for the form slabs during the summer fall, and intermittently during the winter. In the spring of 1949 the latter would be completed, form slabs, pipes and coarse aggregate would be placed and the first step of intrusion performed during August of that year so as to bring the aggregate and grout level approximately to elevation 1710 m or roughly 100 ft above the base by August 31. While the placing of aggregate would continue uninterrupted the second step of intrusion was to be performed during October of 1949 to reach approximately to elevation 1730 m by the end of the month. The placing of aggregate would be continued as long as the weather would permit. No continuous work on a large scale could normally be performed at the site in question between the end of November and beginning of May.

The third step of intrusion was to be performed during June of 1950 bringing the aggregate and grout level up approximately to elevation 1745 m and the fourth and last step during August would complete the dam by the end of the month. The wing dike to the right of the main dam would be constructed separately.

COST

As to the cost of this type of concrete it was fortunate that the same contractor also submitted an independent bid on conventional concrete based on substantially the same time schedule. The latter bid was based on a target estimate prepared by the Morrison-Knudsen office in San Francisco, Calif. The bid on Prepakt concrete was slightly over 5 percent less than the one on conventional concrete. This was in spite of the fact that the Prepakt construction called for 32,500 cu yd more concrete than the conventional concrete construction because of the monolithic treatment of the main dam, that is, the omission of the slots and joints as previously mentioned.

Furthermore the Prepakt proposal provided for a 5-ft parapet wall formed by the upstream form slabs. This would permit a corresponding rise of the reservoir water surface. No parapet wall was included in the bid on conventional concrete.

Discussion of a paper by Paul Baumann:

Use of Prepacked Aggregate Concrete in Major Dam Construction*

By SERGEY STEUERMAN and AUTHOR

By SERGEY STEUERMAN†

Prepacked aggregate concrete was used for the first heightening of the Assouan Dam to join the new and the old masonry.[1,2]

"Cement—Grouting at Assouan Dam Heightening

The main feature of the design is that the new masonry is not initially bonded to the old structure, but is built in such a way as to be supported by it, although separate from it, by steel rods embedded in the old masonry and projecting into the new.

When it is considered that the two masses of masonry have reached suitable temperature conditions, this space will be grouted with cement mortar to make a homogeneous mass.

The space between the new and old work (6 in.) was filled with clean granite chips, and every possible means was taken to prevent spent mortar and dirt getting into the space to be grouted; the wall was carried up a certain height, and then covered over with canvas. As work proceeded, grouting pipes were laid in position. They were 2½-in. galvanized wrought iron pipes, with ¾-in. holes, placed hit and miss on opposite sides, a hole being in each 9 in. of pipe. The pipes were placed 7 m apart in the solid dam, and 3½ m apart above the sluice in the pierced dam. It was found that grouting could not be brought up with safety more than 7 or 8 m at a time. Telltale pipes were inserted 14 m apart horizontally to show that the grouting spread to all parts, and also to allow the water not taken up by the cement to run off."

A later use of the same method—under a new name—for lining a mining shaft with concrete under the water table was described in 1932.[3]

The author, encouraged by the great success of prepacked aggregate concreting for dam repairs on a large scale[4], contemplates its use in the construction of complete new dams. A thorough analysis of the method and a statement of its limitations is justified before it is accepted for structures involving no special features (concreting of narrow spaces or under water, repairs, etc.,) which were given until now as the reasons for the use of the grouting method instead of conventional concreting.

Prepacked aggregate concreting is confronted with the same dual problem as is conventional concreting:[5,6]

*ACI Journal, Nov. 1948, *Proc.* V. 45, p. 229. Disc 45-14 is a part of copyrighted Journal of the American Concrete Institute, V. 20, No. 10, June 1949, *Proceedings* V. 45.

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a. To make the concreting itself a workable process.

b. To produce a set concrete having specific required properties.

CONCRETING

The concreting process with prepacked aggregate is utterly different from the conventional one. In the paper on the restoration of Barker Dam[4] it is stated:

" . . . it is helpful to conceive the Prepakt method of concrete construction basically as the separate handling of the solid (coarse aggregate) and the liquid (mortar) phase. It therefore is comparable with masonry where the maximum size of the sand in the mortar is independent of the minimum size of the stones. Herein lies a fundamental difference between Prepakt and conventional concrete."

The "Two phase" method imposes limitations on both phases:

a. The mortar must be liquid, smooth, and not too quickly disintegrating.

b. The maximum size of the sand in the mortar is independent—in the meaning used for conventional concrete—of the minimum size of the stones, but other limitations on the aggregate grains exist in prepacked aggregate concrete. The aggregate must be permeable to the mortar chosen in a given case, or in general terms, the mortar and the aggregate are correlated—the choice of one predetermines some properties of the other.

MORTAR

Mortar is a cement paste mixed with fine aggregate; liquid mortar can be obtained only with a liquid paste. The properties of the cement paste were described by the writer[5,6] as follows:

"Two W/C ratio's are of special importance to a cement grout:

1. The lowest W/C ratio producing a liquid cement paste made with different portland cements averages .35.

2. The highest W/C ratio producing grouts free from excessive bleeding is equal to .45."

The grains of the concrete (or of the mortar) aggregate passing the 200 mesh sieve do not belong to the aggregate proper but mix thoroughly with the cement to form a binder. The critical values (1) and (2) remain the same, but instead of the cement weight, that of the dry binder, *i.e.*, the weight of the cement plus the weight of all grains passing the 200 mesh sieve, must be considered to establish the "water-binder ratio." The inert grains act as a filler in the binder increasing the strength of concrete having a W/C ratio higher than .35, filling the pores.

If the filler is chemically active with the lime in the portland cement, a further improvement of the concrete properties is achieved. Trass,

pozzolana and granite powder are such fillers and they had been used successfully, especially for concrete in hydraulic structures.

These active fillers increase the stability (diminish the bleeding) of the grout. This stability (dispersion) can be further increased by special admixtures. These "stabilizers" make it possible to use a higher W/C ratio in the grout without producing excessive bleeding and make the grout smoother—they all can be called intrusion help.

The author does not give information about the W/C ratio and the "intrusion agent" used for the prepacked aggregate concrete, but from data given in the paper on Barker Dam[4] the following computations can be made:

1. For the grouting of the foundation rock:

2 sacks of cement	188 lb
Alfesil	100 lb
Intrusion agent	3 lb
Total weight of dry binder	291 lb

Water: Maximum 30 gal.—250 lb Minimum 20 gal.—167 lb

$$\text{W/C ratio} = \frac{250}{291} = .86 \qquad \text{W/C ratio} = \frac{167}{291} = .57$$

2. Grout used for joints less than ¼ in. wide:

Binder—as in (1) Water—14 gal. = 117 lb. $\text{W/C ratio} = \frac{117}{290} = .40$

3. Grout used for joints more than ¼ in. wide:

300 lb of Prepakt sand were added to the binder (2) and the water content increased to 15 gal. or 125 lb. The Prepakt sand contains an average of 5 percent grains passing the 200 mesh sieve, therefore $.05 \times 300 = 15$ lb of fines from this sand enter into the binder; the total weight of the dry binder is:

$$291 + 15 = 306 \text{ lb} \qquad \text{W/C ratio} = \frac{125}{306} = .41$$

Grouts (2) and (3) have a W/C ratio between the limits .35 and .45; in these cases the "intrusion agent" helps only to make the grout smoother.

The grouts (1) have a high W/C ratio; without admixtures they would bleed exceedingly, clogging the pipes and the adjacent rock fissures without reaching the more distant fissures; here admixtures prevent premature bleeding.

AGGREGATE

The tendency of the finest grains of the aggregate to mix with the cement paste due to surface tension limits the minimum fine grain size in the aggregate through which the cement paste or mortar can percolate. To obtain grouting (and avoid mixing), the diameter of the finest grains in the aggregate must be 10 times larger than that of the grout

grains. This is the most important aggregate limitation, especially for sandy and gravelly soils where the soil is the aggregate and cannot be made to order to suit the grouting liquid.

In the writer's experience, grouting is virtually impossible even with increased grout pressure and simultaneous vibration if the above condition is not fulfilled. Increased pressure only speeds up the clogging because the grout mixes with the fines of the aggregate, squeezing the water from the grout.

Another requirement for a uniform permeation of the coarse aggregate is the presence, or formation, of continuous channels in the aggregate. If the aggregate is initially so well precompacted that the voids in the aggregate are not interconnected, the pressure of the rising grout must be above the critical flow velocity (as defined by Prof. K. Terzaghi). At this velocity the aggregate grains are relocated by the grout to form continuous vertical channels in the aggregate.

This procedure is greatly helped by simultaneous vibration of the aggregate (vibroflotation[7]). By this process:

1. During permeation the filter resistance of the aggregate is materially reduced; therefore, the required liquid pressure is diminished and a more uniform permeation of the aggregate with the binder is achieved.

2. The aggregate is compacted during and after permeation. Due to presence of the liquid mortar, forming with the vibrated aggregate an unstable two-phase system, the aggregate is more compact than if compacted in the dry state. After the permeation, the aggregate grains, covered by a thin cement film, touch each other. The void volume in the aggregate and the volume of the binder filling these voids is reduced to a minimum.

PREPACKED CONCRETE

The strength of the concrete is limited by the high W/C ratio necessary to obtain a liquid mortar. For dams and similar heavy structures, the strength of prepacked aggregate concrete is ample. A comparison between the strength of a prepacked aggregate concrete with that of a conventional one of the same W/C ratio shows that the compressive strength of the prepacked aggregate concrete increases in a greater ratio than its tensile strength. This is because the grains of the coarse aggregate touch each other, thus partly transferring the compression directly to each other and not through the mortar.

The principal advantage claimed for prepacked aggregate concrete for dam construction is the reduction or elimination of shrinkage. The author states that this method reduces shrinkage approximately in proportion to the amount of cement used per cu yd of concrete, that is, to ⅓ or ½ that of conventional concrete. He does not analyze the shrinkage process in this special concrete but his comparison of prepacked concrete with stone masonry may explain the shrinkage process of this type of concrete.

Masonry containing the same amount of cement as concrete does not shrink as a unit because the mortar in the masonry does not form a continuous medium in which the stones are "floating," but, on the contrary, the stones form a skeleton in which the spaces between the stones are filled with mortar. In both cases the mortar shrinks, but, in the first, the whole mass shrinks—in the second, only localized shrinkage of the mortar between the stones occurs.

In prepacked aggregate concrete the intrusion must, as shown above, loosen the precompacted aggregate to open the way for the rising mortar and therefore a continuous mortar medium is produced with the resulting partial shrinkage of the concrete.

Vibration compacts the mixture restoring the aggregate skeleton and reducing the void volume in the aggregate and the amount of mortar in the concrete to a minimum. Although this reduction is small compared with the prepacked aggregate concrete, it is a critical shrinkage factor. When vibration is rationally used, the set concrete has an unusually tight structure and its shrinkage is practically eliminated.

In the Barker Dam paper[14] it is stated:

"Ideal distribution of particle size would have resulted in a void content of about 30 percent. From the volume of grout later introduced into the aggregate to produce prepacked concrete, it has been calculated that the actual void content of the aggregate mass was 33 percent."

In this case, vibration would save 10 percent of the mortar and eliminate the residual shrinkage.

The writer knows from his own experience how difficult it is to obtain successful grouting and appreciates the ingenuity of the arrangements proposed by the author for dam grouting. He hopes that his remarks may start a frank discussion of the method by the profession which will bring about a better understanding of the value of prepacked aggregate concrete.

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AUTHOR'S CLOSURE

In his constructive discussion, Mr. Steuerman presents statements regarding certain technological features of prepacked aggregate concrete. The writer concurs in these statements, with certain reservations.

It is well recognized that the shrinkage of mortar, used in the grouting of prepacked aggregate, is, if unrestrained, of the same order as that of regular concrete. However, due to initial contact of the coarse aggregate, shrinkage of the mortar is restrained, resulting in initial tension in the mortar and compression in the coarse aggregate. In other words, a state of prestress is created. Hence, if all of the coarse aggregate were in initial contact, shrinkage would be practically reduced to zero or more exactly to that due to elastic compression of the coarse aggregate only. This, of course, is never the case since initial contact, which is naturally governed by the law of averages, is never perfect.

Vibration of the coarse aggregate, as suggested by Mr. Steuerman, would not change this fact. The placing of the coarse aggregate, if done with the aid of about two cu yd of water to each cu yd of rock, combined with the dumping of the latter, produces a density of the graded aggregate mass which could not be produced nor appreciably improved by means of commercially procurable and economically operable vibrators. Nor would vibration during or immediately after grouting be of much help since, in general, the grouting of the aggregate mass as proposed for major dam construction would take place at such depth as would render vibration applied to the surface ineffectual. It must be remembered that vibration of ordinary concrete is primarily used to expel entrapped air. This is not the case in the proposed construction since the grouting is to displace the water after the latter has displaced the air. The writer thoroughly agrees with Mr. Steuerman, however, that the smaller the void space of the coarse aggregate, the better. On the other hand, he disagrees with him when he states that ". . . the intrusion must . . . loosen the precompacted aggregate in order to open the way for the rising mortar." Since grouting does not have to be carried to the surface except at the crest of the dam, sufficient surcharge of coarse aggregate may always be provided above the grouting level to make loosening thereof physically impossible under the specified pressure conditions.

In discussing fillers, which react chemically with the free lime in portland cement, Mr. Steuerman mentions trass, pozzolana and granite powder. While the first two are noncrystalline or amorphous silicas and are accepted admixtures to accomplish said purpose, granite powder is not a "pozzolan" in the light of strict interpretation of the term. Granite, being largely composed of a crystalline silica, is quite unreactive with calcium hydroxide unless used in the form of an extremely fine powder in which case it will slowly react with calcium hydroxide in the presence of moisture.

Reinforced Concrete Wall and Column Footings*

Part 2

By FRANK E. RICHART†

A SYNOPSIS of this paper, here concluded, was published with Part 1 in October JOURNAL, *Proceedings*, V. 45, p. 97.

TESTS OF COLUMN FOOTINGS, SERIES 3 AND 4

Outline of series

This group of tests, made on footings 7 ft square, supported on a bed of steel car springs and loaded in a large hydraulic testing machine, is similar to the tests of series 1 and 2, described in the preceding part of this report. Properties of the materials, test methods, etc., are all given there.

The test footings of series 3 were planned to study the failure of footings in bond. The design of the footings followed the requirements of the Joint Committee Report,[1] except that allowable bond stresses were exceeded. Since bond was the variable under scrutiny, the footings were designed with both hooked and straight bars. Five different styles of reinforcing bar were used, including billet steel bars having old and new types of deformation, rail steel bars of old and new types, and plain bars.

Bar sizes of $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$ and $\frac{3}{4}$ in. were originally selected in designing the test footings, but as early tests showed no bond failures with the smaller bars, six footings reinforced with $\frac{7}{8}$, 1 and $1\frac{1}{4}$-in. bars were added to the schedule.

With the knowledge that bond failures are difficult to recognize, especial care was taken to measure end slips of a number of bars in each footing tested. The results were characteristically erratic, with some bars showing measurable slips while others nearby showed none whatsoever.

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TABLE 10—PRINCIPAL RESULTS OF COLUMN FOOTING TESTS, SERIES 3 AND 4

All footings 7 ft square, with 14-in. square column, 14-in. effective depth, 16-in. total depth. Average value of f_c' = 3615 psi for series 3, 3365 psi for series 4. Bars in series 4 were welded into mats. Computed values of f_s and u are based on full statical moment. Values of v based on section a distance d from column. Five grades of steel used: NDB is new deformed hard grade; NDR is new deformed rail; PR is plain rail, ODB is old deformed, intermediate grade, ODR is old deformed rail. All bars round, except 1¼ in. square. S indicates straight bars, H, hooked bars.

Footing No.	Compr. strength of concrete f_c', psi	Reinforcement each way: Kind	No.	Size in.	Percent	Yield stress, psi	Max. load, kips	Maximum calculated stress, psi: Tension f_s	Bond u	Shear v	Manner of failure
						SERIES 3					
304a	3430	NDB-S	42	⅜	0.39	66800	524	65600	351	185	DT and T
b	2390						404	50600	269	143	DT
305a	3520	NDR-S	42	⅜	0.39	66100	560	70100	374	198	T and DT
b	3670						558	69900	375	198	T and DT
306a	3750	NDB-H	23	½	0.39	60500 / 69800 *	525	66000	481	186	DT
b	3620						474	59600	434	168	DT
307a	3320	NDR-H	23	½	0.39	74700	520	65400	476	184	DT
b	3810						554	69700	507	196	DT
308a	3820	PR-H	23	½	0.39	60900	536	67400	491	190	T and DT
b	3730						450	56700	412	159	DT
309a	3870	NDB-S	23	½	0.39	60500 / 69800 *	474	59600	434	168	DT and T
b	3570						450	56700	412	159	DT
310a	4110	NDR-S	23	½	0.39	74700	600	75500	549	212	DT and T
b	3920						525	66000	480	186	DT
311a	3620	NDB-H	15	⅝	0.39	55800 / 64600 *	474	59000	532	168	DT and T
b	3410						425	52900	476	150	DT
312a	3630	NDR-H	15	⅝	0.39	63800	512	63700	574	181	DT and T
b	3260						375	46700	421	133	DT
313a	3250	ODB-H	15	⅝	0.39	44800	455	56600	511	161	T
b	3890						468	58200	525	166	T
314a	3200	ODR-H	15	⅝	0.39	70600	550	68400	618	195	DT
b	3880						465	57900	522	165	DT
315a	2880	PR-H	15	⅝	0.39	53200	375	46700	421	133	DT
b	3260						461	57400	519	163	T and DT
316a	3900	NDB-S	15	⅝	0.39	55800 / 64600 *	525	65300	589	186	T and DT
b	4320						500	62200	561	177	T and DT

317a	3610	NDR-S	15	⅝	0.39	63800	486	60500	546	172	DT
b	3720						520	64700	584	184	T and DT
318a	3960	ODB-S	15	⅝	0.39	44800	456	56800	512	161	T
b	3780						472	58700	529	167	T
319a	3800	ODR-S	15	⅝	0.39	70600	475	59100	533	168	DT
b	3500						540	67200	606	191	DT and B
320a	4160	NDB-H	10	¾	0.37	52700	475	62500	667	168	T
b	3275						450	59200	632	159	T
321a	3650	NDR-H	10	¾	0.37	69200	485	63800	681	172	B and DT
b	3830						516	67900	725	183	B and DT
322a	3670	ODB-H	10	¾	0.37	44400	442	58100	620	156	T
b	3970						460	60500	646	163	T
323a	3360	ODR-H	10	¾	0.37	66500	488	64200	685	173	B
b	3760						511	67200	718	181	B, T
324a	3890	PR-H	10	¾	0.37	73000	494	65000	693	175	B
b	4010						504	66300	707	178	B
325a	3720	NDB-S	10	¾	0.37	52700	500	65800	702	177	T
b	3520						475	62500	667	168	B, T
326a	3070	NDR-S	10	¾	0.37	69200	512	67300	719	181	B, T
b	3820						519	68300	728	184	B, T
327a	3940	ODB-S	10	¾	0.37	44400	413	54300	580	146	T
b	3880						375	49300	526	133	T
328a	3080	ODR-S	10	¾	0.37	66500	449	59000	631	159	B
b	3610						450	59200	632	159	B
329a	3640	NDB-S	8	⅞	0.41	49700	489	59000	735	173	T
b	3660						498	60000	748	176	T
330a	3500	NDB-S	6	1	0.40	59700	425	51900	746	150	B
b	3630						425	51900	746	150	B
331a	3705	NDB-S	4	1¼	0.41	59000	450	54400	840	159	B
b	2790						370	43800	691	131	B
						SERIES 4	**welded mats**				
403a	3435	NDB-S	15	⅝	0.39	55800 / 64600 *	440	54800	494	156	DT
b	1960						360	44800	405	127	DT
404a	3305	NDB-S	10	¾	0.37	52700	460	60500	646	163	T and DT
b	3360						360	47300	506	127	DT

*Two qualities of bars, mixed, in lot.

Series 3 contained the usual reinforcing units, consisting of straight or hooked bars, wired together to form a two-way mat. Series 4 consisted of four footings, similar to four of series 3, but reinforced with straight $\frac{5}{8}$ or $\frac{3}{4}$-in. bars, welded together to form an integral two-way mat.

General results of tests

The principal results of the tests of series 3 and 4 are presented in Table 10. Values of the calculated stresses in the table are enlightening as to the cause of failure of these footings. Since the higher calculated steel stresses agreed quite well with the average stresses determined from measured strains, and were generally less than the maximum measured values, it is evident that calculated stresses equal to or even approaching the yield stress are a good indication of initial tensile failure. There was a shearing or diagonal tension failure of all footings containing $\frac{3}{8}$, $\frac{1}{2}$ and $\frac{5}{8}$-in. bars and for some with $\frac{3}{4}$-in. bars. Such failures were evidently secondary where the steel had reached the yield stress, but in general where a diagonal tension failure is indicated in the table, it was an initial or primary failure.

There were apparently no initial bond failures in the footings containing $\frac{3}{8}$, $\frac{1}{2}$ or $\frac{5}{8}$-in. bars, though some slips were noted in nearly all footings before failure. These slips were greatest with the Franklin and the Laclede plain bar. All footings made with Atlantic bars of intermediate grade, having yield stresses below 50,000 psi, failed initially in tension. Among the footings reinforced with $\frac{3}{4}$-in. bars, No. 320 and 325, with Jones and Laughlin hard grade bars, also failed initially in tension, although the computed bond stresses in these footings averaged about 665 psi, and No. 325b exhibited continuous slipping of two bars at failure.

Considering all of the footings made with $\frac{3}{4}$-in. or larger bars in which tension was not the obvious cause of initial failure, it seems probable that failure was due to loss of bond resistance. For these footings the bond stresses ranged from 631 to 840 psi, with an average value of u/f_c' equal to 0.20. Footings 330 and 331, made with 1 and $1\frac{1}{4}$-in. bars, failed unquestionably in bond, developing an average value of u of 756 psi, or $0.22\,f_c'$. For the footings in the group with $\frac{3}{4}$-in. or larger bars which failed initially in tension, the bond value averaged about $0.17\,f_c'$.

The foregoing is admittedly a very rough generalization, since no consideration has thus far been given to the several bar types used or to the end anchorages provided, but it does indicate that in view of current allowable stresses in bond, a satisfactory factor of safety exists.

Maximum bond stresses

An analysis of the values of the nominal bond stress u in the footings having $\frac{3}{4}$-in. and larger bars, has been made from the data of Table 10, and the results are given in Table 11. The bond stress u is here expressed

as a proportion of the concrete strength f_c'. Unfortunately, the yield stress of the Atlantic and the Jones and Laughlin billet steel bars was low enough to produce initial tensile failures, (for ¾ and ⅞-in. bars) and the values of u/f_c' in these cases cannot be considered as representing the full bond resistance of these bars. However, for the rail steel bars and the larger sizes of the Jones and Laughlin bars, the tabulated values are of especial interest. The Laclede deformed bars show an average value of u/f_c' of 0.20, the Franklin bars, 0.19 and the plain Laclede, hooked, 0.17. Values for the 1 and 1¼-in. Jones and Laughlin bars, straight, average 0.22. The showing of the plain and Franklin rail steel bars is surprisingly good in comparison with the new types of deformed bars.

TABLE 11—ANALYSIS OF BOND STRESSES, FOOTINGS 320 to 331

Values listed are for two companion tests except for NDB-¾-in. bars where the companion specimens failed in different ways. All values represent bond failures except those marked with *, which were initial tension failures, and probably did not develop the full bond resistance of the bar.

Kind of bar	Size of bar	Test values of u/f_c' at maximum load	
		Hooked bars	Straight bars
NDR	¾	0.188	0.212
ODR	¾	0.198	0.190
PR	¾	0.177	—
NDB	¾	—	0.189
ODB	¾	0.166*	0.142*
NDB	¾	0.177*	0.189*
NDB	⅞	—	0.203*
NDB	1	—	0.209
NDB	1¼	—	0.237

The foregoing study shows that the new deformed bars of the Laclede and Jones and Laughlin types may be expected to develop a value of the nominal bond stress u, due to full statical load, of 0.20 to 0.22 f_c'. This may be compared with the Joint Committee's design values of $0.0375f_c'$ ($u \leqq 200$ psi) for straight bars and $0.0562f_c'$ ($u \leqq 250$ psi) for hooked bars. However, these design values were chosen for use with a load of only 85 percent of the full load on the footing. Hence, the factor of safety may be computed as the ratio $\frac{0.85 \times 0.20}{0.0375}$, or 4.5 for straight bars and as about two-thirds of this value for hooked bars.

It is to be noted that the foregoing values are nominal bond values, based on the assumption that the bond stress is the same in all bars crossing the standard design section. The measured strains show that this is not true and that the stress in some bars is much higher than the average. The actual bond stresses in some bars are therefore appreciably greater than the nominal values as will be shown later.

Hooked vs. straight bars

The information presented in Table 11 indicates that no consistent advantage has been demonstrated in these tests for the hooked bars, and that the bond stresses developed by the straight bars provide an ample factor of safety. This is surprising and contrary to the belief that has been prevalent among designers for the past two decades. It may be argued that the footings with ¾-in. hooked bars did not develop actual bond failures. In fact, it is hard to define what constitues a bond failure with hooked bars. It has generally been considered that a considerable amount of slipping, while not producing an actual destruction of bond resistance, may be considered an initial failure, since it permits distortion and cracking of the concrete which may precipitate a secondary collapse by diagonal tension. Since such collapses occurred in the footings containing ¾-in. bars, it is difficult to say whether these failures, either with straight or hooked bars, were primarily bond failures, but in view of the large slips measured, they have been so designated. The only tests in which a complete and unmistakable failure in bond occurred were those containing 1 and 1¼-in. bars.

The foregoing comments would imply that if there is any potential value in the end anchorage of good deformed bars, it is not utilized and is not necessary in footings of ordinary conventional design. The footings with straight 1 or 1¼-in. bars, which involved design stresses in bond far beyond anything now permitted, demonstrated an ample factor of safety in bond.

Measured slippage of reinforcing bars

Slipping at the end of both straight and hooked bars was measured in all footings at both ends of three or four bars. These readings were generally unsatisfactory for various reasons. Even though the slip was measured by means of dial gages reading to 0.0001 in., in many cases no slip could be detected on some of the bars, while an appreciable slip was measured on adjacent bars in the same footing.

Besides this fact, slip measurements could not generally be taken at the instant of failure, so that the last set of slip readings was generally taken at the last load increment before failure. A condensed summary of the slip readings for the footings with ¾-in. and larger bars is given in Table 12 and throws some light on the manner of failure of these footings. From this tabulation it should not be inferred that no slipping occurred in the footings made with smaller bars. End slips of several thousandths of an inch were noted in several cases where ⅜, ½ and ⅝-in. bars were used. In the case of No. 315b, made with ⅝-in. plain bars, hooked, several large slips were noted near the maximum load, one as great as 0.029 in. In No. 319a, made with ⅝-in. Franklin straight bars, con-

tinuous slipping on a number of bars was noted and may have produced an initial bond failure.

From Table 12, it appears that the cause of failure listed in Table 10 is as nearly correct as could be determined. Initial failure in No. 320, 322, 327 and 329 was evidently due to yielding of the steel, although some slipping of a few bars had occurred. The failures of No. 323, 324, 328, 330, 331, and probably 326, were evidently due to bond. No. 332, 325 and 326 probably failed because of a complication of causes, including slip of bars, high tensile strains and final collapse due to diagonal tension.

Bond stresses on individual bars in footings

In all of the tests, strain readings were taken along the length of at least one bar located in the central part of the footing width. This provides an opportunity to examine the variation in stress along the bar and thereby to compute the average bond stress at intervals between gage lines for this particular bar. It has been shown that the tensile stress in the various bars crossing the standard moment section varies appreciably from the average value, so that the bond stresses in the individual bar studied may likewise be greater or less than the average for all of the bars. However, for bars in the central part of the footing, the stresses are more likely to be above the average.

TABLE 12—SUMMARY OF SLIP MEASUREMENTS

Measurements of end slip were made on six to eight bars in each footing.

Footing No.	Kind of bars	Max. load, kips	Load at last slip reading, kips	Largest slip, .001 in.	No. bars showing over .001-in. slip
320a	NDB-H	475	425	5.0	3
b		450	450	28.3	1
321a	NDR-H	485	450	9.8	3
b		516	450	6.8	2
322a	ODB-H	442	400	0.6	0
b		460	400	21.1	2
323a	ODR-H	488	450	28.7	6
b		511	450	47.0	4
324a	PR-H	494	450	38.3	5
b		504	450	63.7	3
325a	NDB-S	500	475	23.9	2
b		475	475	—*	2
326a	NDR-S	512	475	—*	3
b		519	450	3.3	1
327a	ODB-S	413	400	2.9	2
b		375	375	10.7	1
328a	ODR-S	449	400	72.8	4
b		450	425	101.1	4
329a	NDB-S	489	475	4.2	2
b		498	475	2.8	2
330a	NDB-S	425	400	40.3	2
b		425	400	8.8	2
331a	NDB-S	450	400	25.4	4
b		370	350	32.3	5

*Slip began at this load on two or more bars and continued until footing failed.

By comparing the actual bond stress in an indivudual bar for a number of footings with the nominal bond stress, $u = \frac{V}{\Sigma ojd'}$, which implies that all bars are stressed alike, one may judge how well the nominal value represents the actual conditions in a footing. Furthermore, the variations noted give some evidence of slipping which takes place in the interior of the footing long before it can occur at the ends of the bar, and if the actual bond stresses are higher than the nominal values, some idea of the potential bond resistance of the bar may be obtained.

Following the foregoing procedure, nominal and actual bond stresses on a number of bars have been determined and are shown in Fig. 12. The values for the footings made with ½ and ⅝-in. bars presumably do not represent the full bond resistance of these bars, but they do show how well the actual and nominal values agree.

Two limitations must be placed on the "actual" bond values. First, they cannot be computed from the measured strains after the yield stress of the steel has been reached; second, they were usually computed on a length of bar extending outward about 14 in. from the column stub. The average bond stress on this length is theoretically only about $\frac{5}{6}$ as great as the maximum value computed at the standard section at the face of the stub, and represented by the nominal bond values. Hence, in computing the "actual" value, found by dividing the change in total tension over the 14-in. length by the surface area of the bar in that length, the result has been multiplied by 1.2 to make it comparable with the nominal values computed at the standard section.

In Fig. 12, bond stress has been plotted against load on the footing. The nominal bond stress is naturally a straight-line function, assuming that *jd* remains constant. The "actual" stress, however, must be low as long as longitudinal tension exists in the concrete, thus reducing the tension in the steel below the nominal value. As cracking progresses, the actual value approaches the nominal value. At the higher loads, in a few cases, the actual value exceeds the calculated value.

The individual curves of Fig. 12 furnish a clue to the performance of the different types of bars. Because of the variability expected of these curves, no great emphasis is placed on them, but it is noticeable that the NDB bars generally show high values and the ODR and PR bars generally give low ones. The latter effect may be interpreted as evidence of extensive slipping with a consequent "leveling out" of tensile stresses within the length of bar under study.

In general, the curves of Fig. 12, while exhibiting a variability to be expected from individual measurements, indicate that the nominal bond

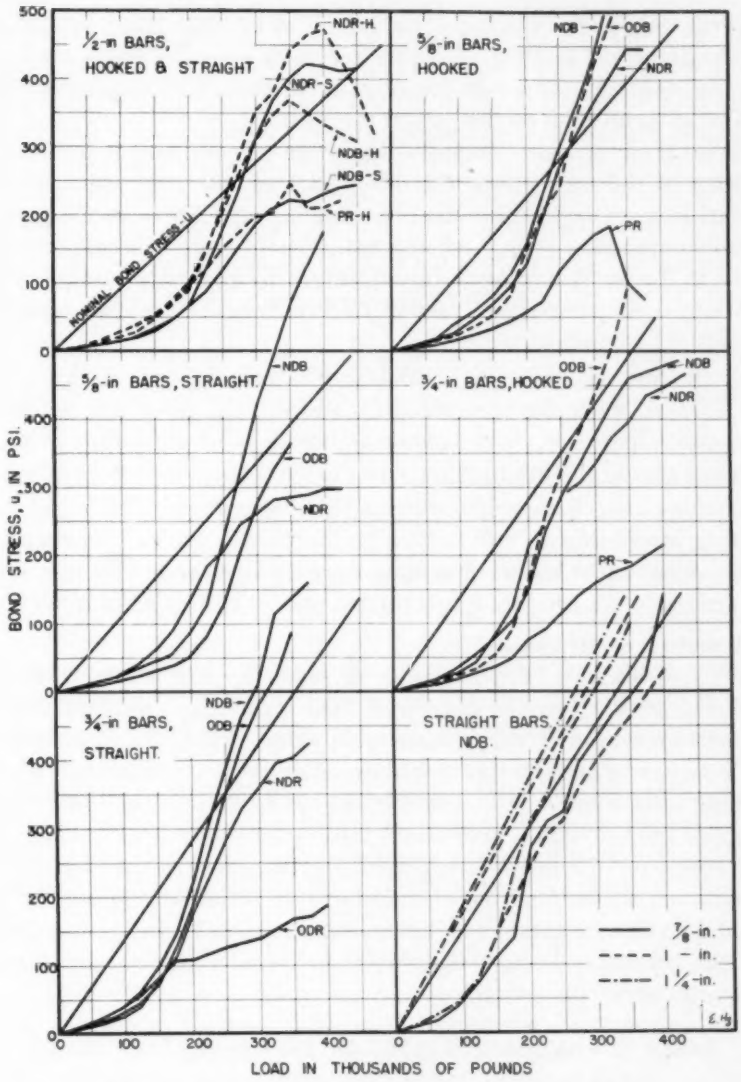

Fig. 12—Comparison of nominal and actual bond stresses

stress, $u = \dfrac{V}{\Sigma\, ojd'}$, may well be used for both interpreting test results and for design purposes.

Resistance to diagonal tension

It is noted in Table 10 that many of the footings of this series failed in diagonal tension, and at somewhat lower shearing stresses than the footings of series 2 which had corresponding effective depths. The only

apparent reason for this situation seems to be that the footings of the current series generally had higher tensile stresses in the steel. Those that did not actually fail initially in tension had calculated tensile stresses of more than 60,000 psi on the average.

These high stresses undoubtedly were accompanied by extensive cracking of the concrete in the lower part of the slab, which would be conducive to an early failure by punching through the remaining concrete in the compression zone. At any rate the values of shearing stress, as shown in Table 13, averaged only 0.049 f_c', by the usual method of computation, as compared with 0.059 f_c' found in the tests of series 2 for footings of 14-in. effective depth.

For the footings of series 3 that failed initially by tension, the value of v/f_c' is 0.045, while for those that failed in bond, the value is 0.047. In these cases, however, it is apparent that initial failure due to another cause prevented the full resistance to diagonal tension from developing.

In none of the foregoing tests was there any appreciable effect on the shearing stress due to end anchorage or to the type of reinforcing bar used. Apparently the steel strains were so large that bond or end anchorage could not produce much further effect on diagonal tension failures

Tensile strains in reinforcement

In the discussion of the results of series 1, it was noted that the measured steel stresses at maximum load for footings that failed in tension agreed very well with the stresses calculated by the usual straight line theory from the full bending moment as obtained from statics. There is ample confirmation of this conclusion in the data of this series. As an example, Fig. 13 shows load-strain curves for a group of footings made with Jones and Laughlin straight bars of various sizes, which failed by tension, or a combination of tension and diagonal tension or bond. The strains are the average values measured on bars crossing the standard moment section. The yield point strain, as determined from tension tests, is indicated on each curve.

TABLE 13—AVERAGE VALUES OF SHEARING STRESS, SERIES 3

Values shown are averages for two tests. Shearing stress, v, is shown as a ratio of the compressive strength f_c'.

Diagonal tension failures				Tension failures		Bond failures	
Footing No.	v/f_c'	Footing No.	v/f_c'	Footing No.	v/f_c'	Footing No.	v/f_c'
304	0.057	314	0.052	305	0.054	321	0.048
306	0.048	315	0.048	313	0.047	323	0.049
307	0.053	316	0.045	318	0.042	324	0.044
308	0.047	317	0.049	320	0.044	326	0.053
309	0.044	319	0.050	322	0.042	328	0.048
310	0.050			325	0.048	330	0.042
311	0.045			327	0.036	331	0.045
312	0.046			329	0.048		

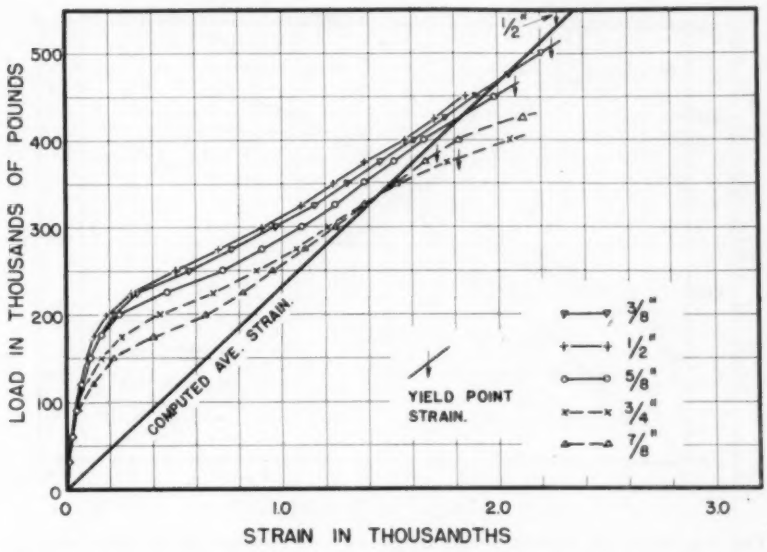

Fig. 13—Average measured and computed tensile strains at standard section, Jones and Laughlin straight bars

Also shown for comparison is the average calculated strain computed from the straight line theory, using the average steel area for the five bar sizes. It is a straight line through the origin of coordinates, since the theory neglects tension in the concrete. It is seen from this diagram that although the strains (and therefore the steel stresses) are much less than the calculated values at low loads, when they reach the yield point, they are in all cases as great or slightly greater, than those calculated by straight line theory on the basis of full statical moment.

Another feature of these curves for which the reason is not clear is the apparent tendency for strains at a given load to be greater for larger bars. This condition is not consistently true for all bars used, but is general enough to warrant mention. It may be that smaller bars are more effective in inhibiting the formation of cracks. At any rate, the effect on the maximum load carried does not seem to be significant.

Fig. 14 presents another grouping of load-strain curves, by which the behavior of various types of bar may be compared. In this figure, average values of strain across the standard moment section for footings made with five types of bar are shown for two sizes of bar, $\frac{5}{8}$ and $\frac{3}{4}$-in. and for both hooked and straight bars. The diagram shows little difference in the curves for the five types of bars. It also shows that the measured stresses at high loads compare very well with those computed from the full statical moment.

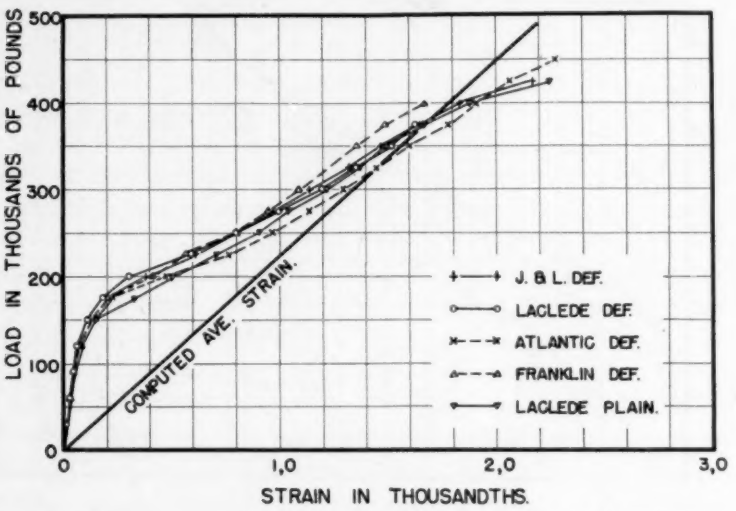

Fig. 14—Average measured and computed tensile strains at standard section, ⅝ and ¾-in. bars of five types

Compressive strains in concrete

Little has been said about the compressive strains and stress in the concrete. From the relatively low steel percentages used in most of the footings, it was assumed that compressive stresses could not have a significant effect on failure, and this has been confirmed by the test results. Strains were measured at a number of locations on the top of the footings just outside the column, and the largest values were generally found across the center line of the footing about 3 in. outside the colunm, as indicated in Fig. 15. The values of compressive strain, measured on a 5-in. gage line, are seen to be generally below 0.001 in. per in. at the highest readings taken. It may be concluded that even at points of highly localized stress, the strains did not approach the usual maximum flexural strain of 0.0015 to 0.002, or more.

Deflections

The deflections of a footing, as measured by a deflectometer on a gridwork of points on the top surface of the footing, furnish information of general interest, as well as a check on the uniformity of the spring reactions against the bottom of the footing. The deflection plotted was the average vertical movement of the column stub, with respect to footing corners. (This deflection was generally a little less than the deflection measured as a check by means of a surveyor's transit and scales mounted on the column stub and the corners of the footing, but the agreement between the two readings was reasonably good.) The load-deflection curves of Fig. 16, are for the footings represented in Fig. 14

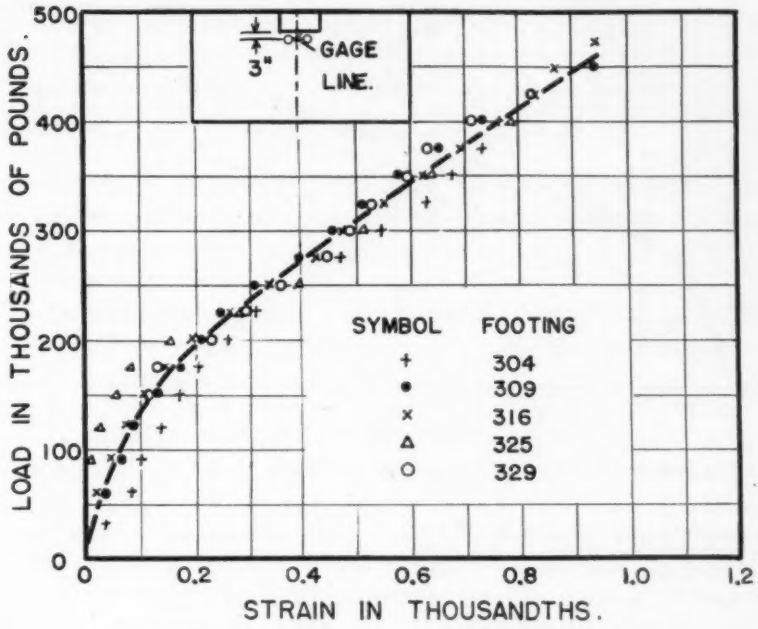

Fig. 15—Measured concrete strains in top surface of footings

and thus should correspond to the strain readings of that figure. However, the deflections show a considerably greater spread in values than did the strains, and in general fall into two groups, with the deflections for footings containing the plain bars and the Franklin deformed bars roughly 25 percent greater than for those made with the other three bars. Since the strains in the entire group were reasonably uniform, this difference in deflection must be attributed to the difference in amount of slip of bars. This conclusion agrees generally with the observed slips of bars, which were markedly greater for the two types of bars mentioned. Thus even the deflection measurements give a definite indication of the inferior bond characteristics of the bars of these two types.

It is to be noted that Fig. 16 includes the results from footings with both straight and hooked bars (except for the plain bars which were hooked in all cases). Average load-deflection curves, not shown here, for the footings having hooked bars and for those with straight bars, were practically identical up to a load of 350 kips. Beyond this load, strangely enough, the deflections of the footings with hooked bars became slightly greater than those for the other group. However, the difference does not appear to be significant. It does, however, strengthen the conclusion reached from considering other aspects of the tests that the hooked ends of bars produced very little beneficial effect in these

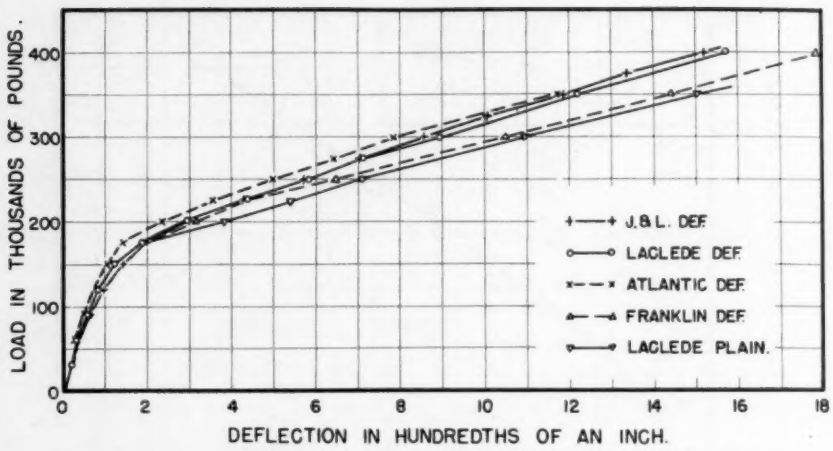

Fig. 16—Load-deflection curves for footings made with $\frac{5}{8}$ and $\frac{3}{4}$-in. bars of five types

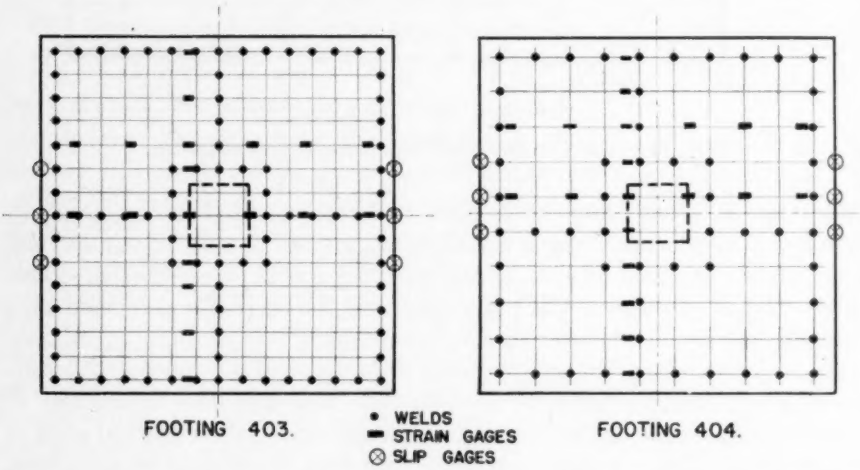

Fig. 17—Sketch showing location of welds in reinforcing mats, also of strain and slip gages

tests. Apparently, the advantages of hooks have been definitely overrated in the current design procedures for footings, with respect to both bond and diagonal tension.

Footings reinforced with welded mats, series 4

This small series consisted of four footings reinforced with welded mats, two containing fifteen $\frac{5}{8}$-in. straight bars each way, and two with ten $\frac{3}{4}$-in. straight bars each way. The bars were welded at the intersections shown in Fig. 17, which also shows the location of strain and slip gages. The results of these tests were intended for comparison with those for footings 316 and 325 of series 3, with straight bars, and 311 and

320, with hooks. However, any comparison with regard to bond is vitiated by the fact that only one footing of the twelve, No. 325b, showed any tendency toward bond failure. All of the others, including the four made with welded mats, failed either by diagonal tension or tension in the steel.

The strains and deflections in footings 403 and 404 fall just about on the average curve for the corresponding footings of series 3.

An examination of the measurements of end slip of bars of footings 403 and 404, which were welded to the transverse bar near the edge of the footing, showed in general very good results. Footings 403b showed no appreciable slip on any of the six gages. No. 403a and 404b showed a slip of about 0.002 in. on one gage only and No. 404a showed slips at two gages of 0.004 and 0.016 in. The latter is the only slip of any consequence and might be due to a poor weld or to the possibility of a crack in the concrete where the gage was attached, which could produce an erroneous reading. With only one slip of any importance indicated out of the 24 observations, it may be concluded that in comparison with the slips observed for series 3 (see Table 12) the welded mats provided exceptionally good anchorage of the reinforcing steel. Welded mats made with straight bars of the improved deformed type should give excellent resistance to bond failure, if this limited series of tests can be considered a sufficient basis of judgment.

Summary

The footings of series 3 which failed by bond, or in which bond was evidently a contributing cause of failure, furnish the basis for the following conclusions:

1. For the footings tested, there were no failures definitely attributed to bond stress in footings containing $\frac{5}{8}$-in. or smaller bars. The footings made with $\frac{3}{4}$, $\frac{7}{8}$, 1 and $1\frac{1}{4}$-in. bars that did not fail initially by passing the yield point of the steel in tension generally failed in bond. Even in this group, the footings made with $\frac{3}{4}$ and $\frac{7}{8}$-in. bars finally collapsed by a typical diagonal tension punching failure, though this was definitely a secondary failure.

2. Of the types of bars used, the deformed bars of the new improved type (Jones and Laughlin billet and Laclede rail steel) gave superior results. The Atlantic bar, which appears to have one of the better of the older types of deformation, unfortunately was used in intermediate grade billet steel, which resulted in tension failures of all footings made with these bars. The Franklin rail steel bars, having a longitudinal type of deformation, and the Laclede plain bars, gave inferior bond resistance, although better than had been expected.

3. For the footings classed as bond failures, bars provided with standard hooks at the ends showed no advantage in bond resistance over straight bars. The hooks did not prevent end slips of appreciable amount, particularly with the Franklin deformed and Laclede plain bars.

4. Compared with bond stress computed from the change in tensile stress in

individual bars, the full nominal bond stress, $u = \frac{V}{\Sigma\, ojd'}$, seems to give a good measure of bond resistance in column footings.

5. Maximum values of the nominal bond stress in footings reinforced with improved deformed bars ranged from 0.19 to 0.23 f_c' on straight bars.

6. Maximum values of shearing stress, developed in these footings having a 14-in. effective depth, were generally lower than those found in comparable footings of series 2. This may be due to the fact that the steel stresses were high and extensive cracking of the footings of series 3 had developed before diagonal tension failures occurred.

7. The steel strains measured in these footings confirm the findings in series 1 which show that the steel stresses correspond to the full bending moment as found by statics.

8. Measurements of deflections of footings apparently confirm the observation that extensive slipping occurred in footings containing plain bars and those having longitudinal deformations.

9. Welded mats made with Jones and Laughlin deformed bars were effective in minimizing the slip of reinforcing bars.

TESTS OF RECTANGULAR COLUMN FOOTINGS, SERIES 5

General results of series 5

As mentioned under "Theoretical calculations" at the beginning of Part 1, the rectangular footing presents two interesting questions to the designer. One relates to the distribution of bending moment across a standard section running along the face of the column and lengthwise of the footing. The other concerns the method of computing the shearing stress, v. The ACI Building Code Requirements[2] provide arbitrary rules governing both matters, but no accurate theoretical solution is available for either problem. This group of tests was made in the hope of providing new information on the subject.

Tests were made on six footings 6 x 9 ft in plan and six footings, 5 x 10 ft in plan, all having a 14-in. square column stub in the center. In each group two thicknesses of slab were used and, as a further variable, welded mat reinforcement was used in two footings of each group. The footings were tested in the same manner as the preceding column footings. A view of one of the footings in the testing machine is shown in Fig. 18.

The principal results of the tests are given in Table 14. All of the footings failed by diagonal tension, though it is evident from the strain measurements that in all of the thick footings, most of the long reinforcing bars had first yielded. Since the footings had been designed by use of the ACI Code procedures, it is not surprising that the calculated tensile and bond stresses are fairly uniform in the two directions. In shear, however, no such equalization was possible.

Tensile strains and stresses

Fig. 19 shows plotted values of measured strains across the standard design section for footings 502 and 504. Footing 502 had a 16-in. effec-

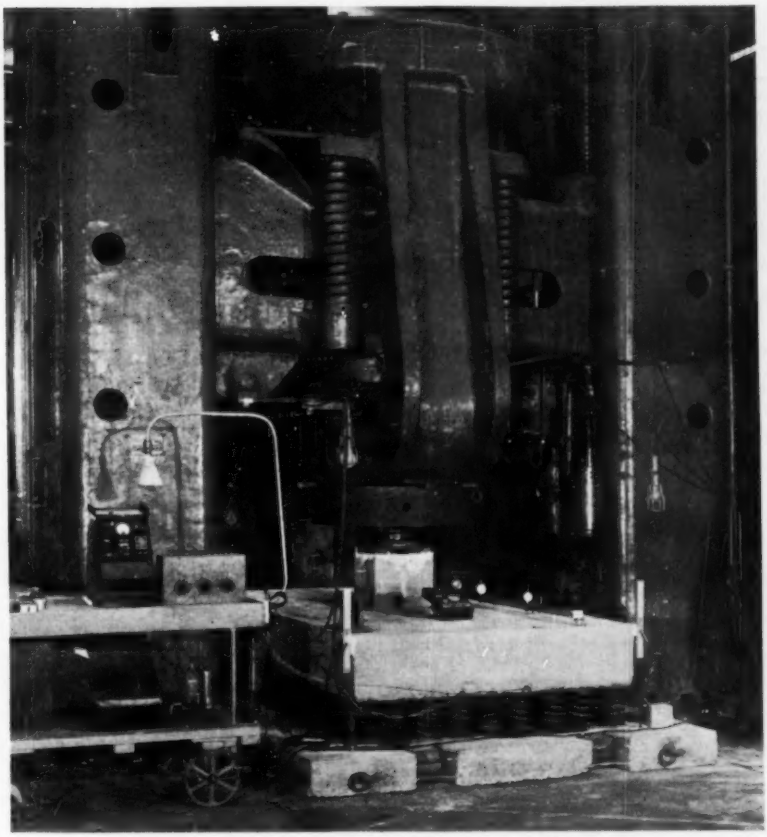

Fig. 18—View of rectangular footing in testing machine

tive depth, and like all of the footings of this depth, developed the yield stresses in the long bars before the footing failed. The distribution of stress in the long bars was fairly uniform at loads of 100 and 200 kips. At 300 and 400-kip loads, the strains became greater in the central portion of the footing width, while at a load of 500 kips, several of the bars had reached the yield stress, and evidently all of them did before failure of these footings.

The stresses in the short bars of the same footings are of interest, though of less importance. The diagram shows a decided concentration of stress in the bars near the center of length at all loads. Some of these central bars evidently reached the yield stress before the footings failed at loads of 554 and 578 kips, respectively.

Footing 504 had an effective depth of 10 in. and was 5 x 10 ft in plan. The tensile strains in these footings in no case reached yield point values, which were unusually high. As in footing 502, the strains were reasonably

TABLE 14—RESULTS OF RECTANGULAR FOOTING TESTS, SERIES 5

Footings 501 to 503 were 6 x 9 ft; footings 504 to 506, 5 x 10 ft. All had columns 14 in. square. Effective depth 10 in. for footings 501 and 504; 16 in. for all others; 2 in. cover. Steel, Jones and Laughlin hard grade deformed billet. Average value of $f_c' = 3600$ psi. Bars in footings 503 and 506 were welded into mats. Computed values of f_s and u based on full statical moment. Values of v based on section a distance d from column. Long way of footing indicated by L, short by S. Bars hooked at ends except in footings 503, 506.

Footing No.	Compr. strength of concrete, f_c', psi	Reinforcement			Yield stress, psi	Max. load, kips	Maximum calculated stress, psi			Manner of failure
		No.	Size	Percent			Tension f_s	Bond u	Shear v	
501a	3690	32	⅝-L	1.38	61600	394	45100	303	381	DT
		52	⅜-S	0.53	76900		46100	287	167	
501b	3725	32	⅝-L	1.38	61600	380	43500	292	368	DT
		52	⅜-S	0.53	76900		44500	277	161	
502a	3530	31	½-L	0.54	60900	554	63400	341	222	T and DT
		32	⅜-S	0.20	76900		63800	408	82	
502b	3285	31	½-L	0.54	60900	578	66200	356	232	T and DT
		32	⅜-S	0.20	76900		66600	426	86	
503a	3545	31	½-L	0.54	60900	586	67100	361	235	T and DT
		32	⅜-S	0.20	76900		67500	432	87	
503b	3480	31	½-L	0.54	60900	550	63000	339	220	T and DT
		32	⅜-S	0.20	76900		63400	405	82	
504a	3605	24	¾-L	1.77	74000	325	41100	282	356	DT
		37	⅜-S	0.34	76900		43200	318	90	
504b	3735	24	¾-L	1.77	74000	350	44300	303	383	DT
		37	⅜-S	0.34	76900		46500	342	97	
505a	3680	21	⅝-L	0.68	61600	548	68400	408	249	T and DT
		22	⅜-S	0.13	76900		69300	559	43	
505b	3730	21	⅝-L	0.68	61600	525	65500	391	239	T and DT
		22	⅜-S	0.13	76900		66400	536	41	
506a	3350	21	⅝-L	0.68	61600	500	62400	372	228	T and DT
		22	⅜-S	0.13	76900		63200	510	39	
506b	3815	21	⅝-L	0.68	61600	500	62400	372	228	T and DT
		22	⅜-S	0.13	76900		63200	510	39	

uniform for the long bars of these footings, but were high near the center of the footing in the short bars, and very low near the ends.

In studying the action of the reinforcing steel in these footings, load-strain curves were plotted for footings 502, 503, 505, and 506 in which the steel stresses reached the yield stress. Fig. 20 shows average curves for each group of footings that were alike in design. The curves for long bars are similar to those for rectangular footings. It is evident that the long bars in each case reached the yield stress before the footings failed.

The curves for the short bars are quite different in appearance . This is evidently because of the wide spread in the stresses across the standard moment section, as shown in Fig. 19. Although these average measured values are much less than the computed ones, shown by straight lines, an examination of the invididual strains shows that the maximum strain on short bars near the center of each of these footings reached the yield stress at or near the maximum load.

The concentration of the lateral bending in these footings into a fairly small area near the middle is understandable from the nature of the curvature of the footing. Thus, if a short rigid beam had been inter-

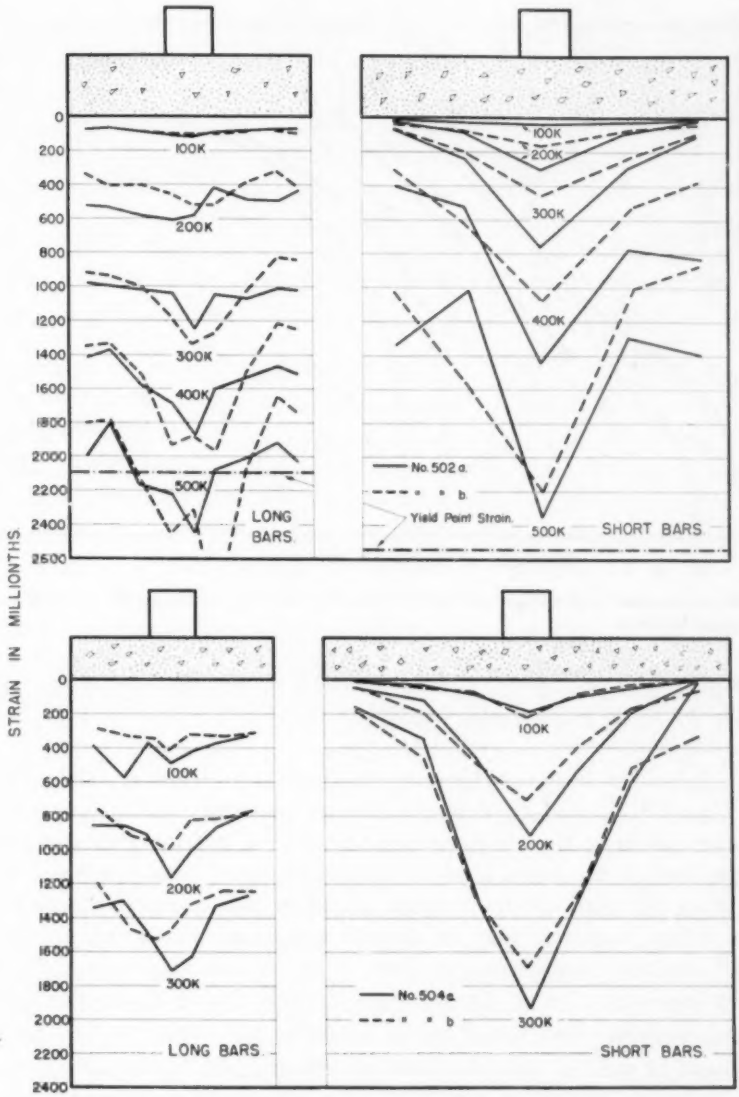

Fig. 19—Measured tensile strains across standard moment section, rectangular footings

posed between the footing and the column, it would have received practically all of the lateral bending moment, and by minimizing the lateral curvature, would have relieved the slab from most of the lateral stress and made it act, in effect, as a one-way footing. The lateral reinforcement directly under the column evidently serves the function of such a beam, but is insufficient to prevent large curvature and the

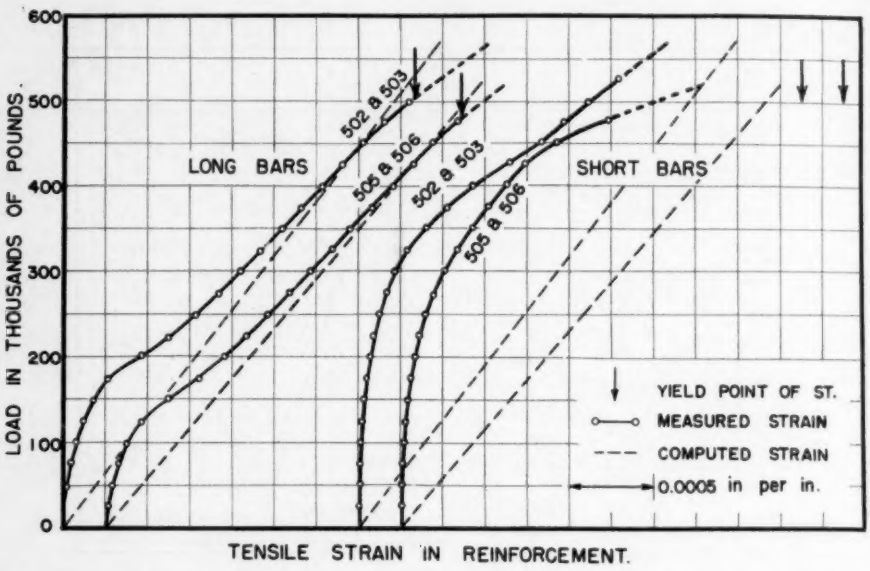

Fig. 20—Measured and computed average tensile strains across standard moment section, rectangular footings

attendant stresses. However, the lateral curvature near the ends of the footing is very small and hence the stresses in the short bars in that region are small.

This behavior of the rectangular footing justifies the provision of the ACI Code[2] (followed in designing these footings) which provides for a concentration of the short bars in the central portion of a rectangular footing, but it indicates that considerably more concentration of the short bars adjacent to the column than the code requires would be still better. The question may be largely academic, since the amount of lateral reinforcement is small, but it is evident that a considerably greater concentration of lateral steel directly under the column could be used without developing undue compressive stress in the concrete.

The use of welded mats in footings 503 and 506 gave perfectly satisfactory results, but the results are not particularly significant as to the effectiveness of the welded intersections as anchorage for the reason that the bond stresses in these footings were not great enough to cause any difficulty. The location of the welds were similar to that shown in Fig. 17 for series 4, with the addition in these rectangular footings of a solid row of welds across the footing at the ¼-points of the length.

Shearing stresses

Table 14 shows that rather high nominal shearing stresses were developed in these rectangular footings. For the footings having a 10-

in. effective depth, which definitely failed in diagonal tension, the average shearing stress was 372 psi, or $0.10f_c'$. For the footings having 16-in. effective depth the corresponding values are 231 psi, or $0.065f_c'$. These are the shearing stresses on a section normal to the length of the footing; the stresses in the other direction were very much smaller.

The shearing stresses just mentioned were computed by the procedure of the ACI Building Code Requirements,[2] outlined at the beginning of this paper. They are definitely greater than the values found from the tests of series 2, in which footings with a 10-in. effective depth gave average shearing stresses of $0.07f_c'$ and those with 16-in. effective depths gave average values of 0.053.

An explanation of the rather high shearing stresses in these footings may be found from a study of the manner of failure. Three types of diagonal tension failure were observed in this group of footings: (a) the ordinary punching failure, (b) failure by an inclined crack extending across the full width of the footing, as in a beam, and (c) a combination of (a) and (b). All of the footings of 10-in. effective depth failed according to type (a). Of the thicker footings, in the 6 x 9-ft group, one failure was of type (a), one of type (b) and two of type (c). Of the thicker 5 x 10-ft footings, three failures were of type (b) and one of type (c).

These observations point to an obvious conclusion. As the proportions of rectangular footings are varied from the square footing to a long, narrow footing, the manner of diagonal tension collapse will change from a punching failure to a beam-type failure. The footings of this series were evidently in the transition zone. Even when a punching failure resulted, if the shearing stresses were much greater on two sides than on the other two, it is likely that there was some departure from the conventional effective widths of section. When the failure was of the beam type, it is easily demonstrated that while the vertical shear acting was increased slightly, the width resisting failure was increased by one-third to one-half. The point of this discussion then, is that in rectangular footings, some departure from the conventional punching failure is to be expected and since the conventional section resisting diagonal tension is logically smaller than the actual one, values of the conventional shearing stress may be expected to be abnormally high in comparison with results for square footings.

Summary of rectangular footing tests

Two or three features of these tests may be emphasized:

1. The average stress in long bars in rectangular footings agreed very well with that calculated from the full static bending moment at loads approaching the ultimate capacity of the footing.

2. The stress in the short bars in such footings varied greatly from middle to end of footing. The average stress was generally less than the conventional calculated value. There is evidently a sound basis for concentrating the short

bars in the central part of the footing to a greater degree than that prescribed in the ACI building code.

3. Maximum shearing stresses in these footings were definitely greater than those developed in square footings. This suggests that in these footings a somewhat greater width of shearing section was effective than that prescribed in the ACI design procedure. It seems logical that as the ratio of length to width of footing increases, the failure section must change from a pyramidal punching surface to the usual inclined diagonal plane across the width of the member, as in a beam.

TESTS OF FOOTINGS WITH CAPITALS, SERIES 7

General outline and test results

This small series of four tests was made to observe the effect of a column cap having a width one-fourth that of the footing slab. The footings were 7 ft square, with a plain concrete capital 21 in. square and 12 in. deep, surmounted by a 14-in. square column. Two effective depths were employed, 8 in. and 12 in., and these footings were intended for comparison with footings 207 and 209 of series 2. The capital was utilized in designing these footings only to the extent of modifying the amount of reinforcement required. A view of one of the footings in the testing machine is shown in Fig. 21. The principal results of the tests are given in Table 15. Failures were by diagonal tension except in No. 702b, which evidently failed initially in tension.

Discussion of results

Although the footings with capitals were made with concrete a little weaker than that in footings 207 and 209, it is evident that the former

TABLE 15—RESULTS OF TESTS OF FOOTINGS WITH CAPITALS, SERIES 7

Footings 7 ft square with capital 21 in square. Bars, ½-in. round, Jones and Laughlin hard grade deformed, ends hooked. These footings designed for comparison with footings of series 2, without capitals. All failures were apparently by diagonal tension except 702b, which failed initially in tension. Tension and bond stresses computed from full load, shearing stress from use of section a distance d outside column or capital.

Footing No.	Eff. depth, in.	Compr. strength of concrete, f_c', psi	Reinforcement, ½-in. bars each way		Yield stress, psi	Max. load, kips	Maximum calculated stress, psi			Remarks
			No.	Percent			Tension f_s	Bond u	Shear v	
701a	8	3860	33	0.98	60900	434	53900	436	328	With cap
b	8	3545	33	0.98	60900	448	55700	450	338	With cap
207a	8	4250	42	1.25	60500*	360	43400	315	364	Without cap
b	8	4040	42	1.25	69800	342	41200	300	345	Without cap
702a	12	2120	22	0.43	82900	445	55300	447	163	With cap
b	12	2885	22	0.43	60900	520	64600	522	191	With cap
209a	12	2890	27	0.54	60500*	440	55000	400	213	Without cap
b	12	2355	27	0.54	69800	420	52500	382	203	Without cap

*Mixed lot of bars of two qualities.

Fig. 21—View of footing with capital in test machine

required only about four-fifths as much reinforcement and developed maximum loads on the average about 19 percent greater. The tension and bond stresses were very nearly proportional to the maximum load in the two types of footing. The maximum shearing stresses, even with average loads 19 percent greater for the footings with a capital, were about 11 percent smaller than for the footings without capitals.

Since diagonal tension is of principal concern in these tests it may be noted that values v for these footings were as follows:

Footing No.	d, in.	v, psi	v/f_c'	Remarks
701	8	333	0.090	With capital
207	8	355	0.086	Without capital
702	12	172	0.072	With capital
209	8	208	0.082	Without capital

It is evident from this comparison of shearing stresses that there is no particular advantage for either type of footing in the value of maximum shearing stress developed.

CONCLUSION

General summary of footing investigation

Since the results for each series of tests have been summarized in some detail, only the most important conclusions will be repeated here:

1. The stresses in the reinforcing steel, derived from the measured strains, agreed very well near the yield stress with the values of stress computed from the full statical bending moment acting on the footing. While there was a marked variation in tensile strains in adjacent reinforcing bars, without much consistency in pattern except that center bars on the average had the higher strains, it is evident that in footings that failed by tension, all of the bars reached the yield stress and hence were about equally effective.

2. To be consistent with the tensile stresses computed from the full moment, the bond stresses have been computed from the full load on the footing. On this basis the bars with new improved types of deformations developed maximum bond stresses of 0.19 to 0.23 f_c'. Bars with ends hooked showed no apparent advantage over straight bars as to maximum bond stress developed, and very little difference as to the end slip of the bars. Welded mats appeared to be very effective, especially as to end anchoarage. Plain bars and bars with longitudinal deformations were considerably less effective than the improved bars, either as to maximum bond stress or as anchorage against slipping.

3. A very large number of the footings tested resulted in diagonal tension failures. The shearing stresses obtained, as calculated in the conventional way at a distance d from the face of the column, varied generally from less than $0.05f_c'$ to $0.09f_c'$. The value seemed to increase consistently as the effective depth of the footings decreased. Most of the footings which failed initially due to tension, or excessive bond slip, collapsed finally by a diagonal tension failure, with a pyramidal plug of concrete punching through the slab beneath the column.

The increase in shearing stress produced by hooks at the ends of bars was generally small, and frequently absent entirely. Apparently the effect of end anchorage, as currently recognized in allowable stresses for both bond and shearing stress, has been greatly over-rated.

The effect of extensive cracking of the footing due to high tensile stress, and probably to slip of bars in the interior of the footing, was evidently to cause early failure by a diagonal tension collapse.

4. The behavior of rectangular footings did not differ greatly from that of square ones. The main bars running lengthwise of the footing behaved normally, but the stress in the short bars was highly concentrated at mid-length of the footing. Maximum shearing stresses, computed by the ACI Code procedure, were generally higher than those found in square footings. The test footings at maximum load were evidently in a transition stage between a punching failure, as in a slab, and a diagonal failure across the width of the member, as in a beam.

5. The use of a capital or pier between footing and column was found to be effective in saving reinforcement and in increasing the load-capacity of the footing. The results were in good accord with design theory.

6. In general the investigation, has covered a range of materials and designs, for which experimental verification was largely lacking. Some of the results were very much unexpected. For example, it seems that shearing stress rather than bond stress may frequently be the critical feature of the design of a footing. In the preparation of this report there has not been an opportunity to make a complete study of the data accumulated, and what analyses and interpretations of results have been made were aimed to furnish a concept of the structural action of the test numbers, rather than as recommendations for design. With further study of the original data and with this report available for general study, it seems likely that worthwhile improvements in present design procedures for footings will be forthcoming in due time.

In conclusion, the writer again wishes to express his appreciation to the Committee on Reinforced Concrete Research of the American Iron and Steel Institute for having made this investigation possible.

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2. "Building Code Requirements for Reinforced Concrete," American Concrete Institute, 1947.

Discussion of a paper by Frank E. Richart:

Reinforced Concrete Wall and Column Footings*

By WILLIAM L. SHANNON, ROSS H. BRYAN and AUTHOR

By WILLIAM L. SHANNON†

It is interesting to note that in the tests of concrete footings the reaction against the bottom of the footings is made as uniform as possible by the use of springs, as shown in Fig. 3 of the paper. During the tests, the springs deflected approximately the same amount until the yield stress of the steel reinforcement was reached, then those springs under the central portion of the footing deflected more than those at the exterior of the footing, and finally under large loads the springs under the central portion closed. Thus, at the beginning of loading of a given footing, the reaction pressure was essentially uniform until the yield stress in the steel was reached, beyond which the pressure distribution gradually changed until it was an indeterminate amount greater in the center than at the edges.

It is interesting to compare the test conditions with those which are likely under actual loading of a footing resting on soil. Let us assume that a homogenous and isotropic soil underlies the footing and that the footing is rigid. If the footing rests on the surface of a noncohesive soil, such as sand, the soil pressure on the footing will vary from nearly zero at the edges to a maximum at the center. A deep footing, like a bridge pier, on a noncohesive soil such as sand, will have a soil pressure which will be practically uniform over the entire footing. A footing on a cohesive soil, such as a clay, will have a soil pressure which will be large at the edges and small at the center.

Natural deposits of soil are very infrequently homogeneous and isotropic. Soil conditions beneath footings may be such that the pressure distribution on the underside of the footing will be much greater on one edge than on the opposite edge. Further, treatment of the foundation of a footing, by adding gravel to a soft spot or compacting local areas of loose soil may result in an appreciably unbalanced pressure distribu-

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tion. Thus, a uniform soil pressure on the underside of a footing frequently does not result.

Since the tests were all performed using an essentially uniform soil pressure, application of the results of these tests should be limited to similar site conditions. It appears reasonable to design buried footings on noncohesive soils, such as sand or sand and gravel, using criteria determined from these tests.

It is hoped that the author will perform tests in which the pressure is much greater at the edges than at the center and tests in which the pressure is greater on one edge than on the opposite edge. Such tests should provide information essential for the design of footings on cohesive soils and on heterogeneous soils, both of which may produce pressure distributions far more critical from the standpoint of footing design than the uniform pressure distribution tested.

By ROSS H. BRYAN*

All footings tested were apparently loaded by the reactions of a bed of coil springs which gave approximately equal reactions up to the critical loads where the deflection at the edge of the footing was approximately 0.10 in. The approximate load per sq ft at the critical load was about 9000 lb—a fairly common design pressure for firm soil.

It should be noted that for a footing bearing on firm soil a deflection of 0.10 in. at the outer edge of the footing would relieve the pressure in this area considerably and result in a bulb of pressure being formed under the column pedestal. The shape of this pressure curve has been fairly well established by various investigators. If the above statement is accepted then it follows that the moment at the face of the column for a footing having a bulb-shaped pressure curve will be less than the moment at the face of the column for a footing having a uniform pressure. It would seem then that the footings which failed by tension in the tests might, if tested under actual loading conditions, fail by bond or diagonal tension.

The results of the tests seem to indicate that the value of a hook in good concrete is not as great as most codes have considered it to be.

AUTHOR'S CLOSURE

It is interesting that the discussions by both Mr. Shannon and Mr. Bryan are concerned with nonuniform soil pressures to be expected under a footing in service and the probable effect of such conditions

*Structural Engineer, Marr & Holman, Architects, Nashville, Tenn.

upon the stresses in the footing. In planning this series of tests, it was recognized that such pressures may exist in the field, producing sometimes more, sometimes less, severe moments and shears than the uniform load commonly assumed. However, to have carried on even a small fraction of the test program in the field would not only have been exceedingly expensive and difficult, but the measurement of the actual soil pressures would have been the major problem. The pressure pattern would probably have been different for each test, so that any comparison of the test results would have been almost impossible. Our object, in employing essentially uniform bearing pressures, as produced by the bed of coiled springs, was to employ a known set of loads and reactions, which could be applied with high precision to a large number of footings and which would permit the rapid determination of the corresponding moments and shears at any stage of the test. We had planned a few tests with nonuniform spring reactions, but these were finally omitted from the schedule in favor of other tests thought more important.

It seems reasonable that if we can determine the behavior of a footing due to moments and shears produced by uniform reactions, we will not be greatly in error in expecting similar behavior under given moments, or shears, produced by any probable pattern of nonuniform reactions.

Referring to the most unfavorable distribution of soil reactions, with the highest pressures near the edges of the footing, it seems logical that the high values may be reduced relatively by deflection of the footing under load, and in some cases, by flow of the soil. However, it cannot be denied that conditions will sometimes occur which are less favorable than the design assumption involving uniform pressures.

Regarding Mr. Shannon's comment on the deflection of the springs and their final closing up solid, it should be noted that this complete closing occurred only in the tests of wall footings where the deflection of the projecting slabs was excessive. In none of the column footings did the springs close up at any stage of the tests. The departure from a perfect uniformly distributed reaction on the bottom of all column footings, due to deflection of the footing, was negligible until after the yield point of the tensile steel had been exceeded, and initial failure had occurred.

Letters from Readers takes over from the former **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present ideas (the most valuable things we have) for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

A. A. LEVISON, L. G. HILKEMEIER, ROBERT F. PORTER, C. GRAY and E. L. HOWARD tell of their experiences in eliminating cement balls in concrete—continuing consideration of the question asked by a correspondent and discussed by Stanton Walker in the October Journal.

Cause and Cure for Cement Balls in Concrete (LR 45-8)†

A. A. LEVISON of Blaw-Knox Co., Pittsburgh suggests methods of control

If the aggregates are being charged into the mixer drum separately, as is often the case with truck mixer operations, the coarse aggregate should be charged into the drum first, then followed by the fine aggregate, with the cement last. We have found in some cases, where the sand and cement are introduced first and the coarse aggregate last, that balls of cement and sand are formed and are not broken up by the mixing action.

In the case of a stationary mixer or a truck mixer where all materials are loaded into the mixer drum more or less simultaneously, it is frequently possible to have the coarse aggregate lead the procession of materials into the drum, which is advantageous.

Whenever possible a portion of the mixing water should be introduced in advance of any materials, to wash off the blades and the interior of the drum. A good rule is that up to one-third of the mixing water should be in the drum before the aggregates and cement are introduced.

With truck mixer operations where no mixing is done until the truck mixer arrives at the job it is best to introduce the cement into the mixer

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 3, Nov. 1948, *Proceedings*, V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

†Question originally presented in ACI JOURNAL, Oct. 1948, p. 181, by R. D. Rader.

drum as the last material and have the cement form a layer over the top of the aggregates. If the cement is intermingled with the aggregates and no mixing takes place until the unit arrives at the job balls of cement or cement and sand may result, especially on a long haul.

The interior of the mixing drum and the blades should be kept clean and free from hardened cement; worn mixing blades should be repaired or replaced to maintain mixer efficiency.

L. G. HILKEMEIER of Chain Belt Co., of Milwaukee tells of successful remedies in truck mixer operations

I will only relate some of the contributing factors to balling and what we did to remedy the complaints in the field.

We have in the past made investigations of field complaints where balling occurred and each time one of the following items was found to be the cause: 1) hot cement, 2) hot aggregate or water (winter operation), 3) loading sequence of material into mixer, 4) excess wear on mixer blades.

Hot cement: Of the four items mentioned the first three were predominant; in fact, "hot cement" will produce balls in sizes from ⅛ in. in diameter to 6 in. In a case not so long ago we encountered cement having a temperature above 180 F which produced balling in the range stated above. We were able to eliminate the larger balls from 2 in. and up by changing the loading sequence so that the water would not trail the coarse aggregate and sand, thereby eliminating pools of water that formed, this in turn prevented the hot cement from coming in contact with the water where flash set took place.

Hot aggregate or hot water as used in winter operations: In this case flash set will occur if cement is allowed to come in contact with an over-wet hot zone in the mixer and as a result cause balling of cement, sand and stone. This also can be over-come by having aggregate and sand trail the water.

In a recent case, an operator reported balling and in checking his plant we found that his water measuring unit was installed in the cement batch plant, separate from the aggregate plant. Three suggestions were offered: 1) add water to aggregate before loading cement, 2) move water measuring unit to aggregate plant or 3) run water through the truck mixer water system.

Excess wear on mixer blades: As a conservative yet adequate wearing limit we recommend a maximum average height reduction of 15 percent for blades which are otherwise in good condition. Balling in this case is of a different nature. Balls of unmixed sand and cement with no appearance of quick set may be formed which, because of the worn blades, may be laid to insufficient mixing.

Another problem in ready-mixed operation is to obtain a grout mix for topping without "balling." This has been done successfully by mixing the sand and cement for about 20 revolutions of the drum before the water is added.

ROBERT F. PORTER, of Baltimore, Md.

Last year, we were having trouble with cement "balling up" in our mixers, and after checking every angle, we found that 90 percent of the trouble was from hot cement. We also found that the position in which the paddles were placed in some types of mixers caused the cement to ball. After finding the trouble, we let the water enter the mixer slowly and since then we have had very little trouble.

This problem caused us much anxiety, and a number of us entered the study of it with the result that we have eliminated it in our operations.

C. GRAY, President, Ready Mixed Concrete Corporation, Indianapolis, Ind., writes:

Our experience with lumps or balls of cement appearing in concrete after apparently having been well mixed has been confined to the richer mixes of a low slump. It is my opinion, that "balling" is caused by feeding the cement in too rapidly. I do not believe that it would occur in mixtures with a fixed cement content up to and including 8 bags, provided the flow of water was started into the mixer first, the sand and gravel then ribboned in, and the flow of the cement then started in a ribbon, falling on the sand and gravel as it entered the mixer.

We feed cement into the mixers through a rectangular closed chute not over 24 sq in. cross-sectional area which runs from the bottom of the weigh-batcher into the sand and gravel chute which enters the mixer. This closed cement chute ends about 2 in. above the top of the gravel stream. When the cement is fed in very rapidly, in a large volume, the water penetrates the outer surface of the stream of cement, forming a plastic ball, with dry cement inside. I have opened up these balls as soon as the concrete was mixed and have found absolutely dry cement in the centers. After the concrete stands for a while the water penetrates to the center of the ball and the entire mass or ball becomes plastic, but does not dissolve to the point where it will flow and break up unless the concrete has a high slump and contains a large amount of water.

The above opinion is based on my experience with the few occurrences of this nature that we have had.

E. L. HOWARD, Testing Engineer, Pacific Coast Aggregates, Inc., San Francisco, Calif.

My experience has been that the first cause to examine when troubled with lumps or balls of cement in the concrete mix is the mixer and mixing facilities. The second cause could be the manner of charging the mixer.

We have had no lumps in concrete at those plants where the cement and aggregate are charged into the mixer together and partially mixed before contacting water. We have experienced lumps in those plants where old mixers are used, in which the blades are in bad shape, or where the mixer was overcharged. That is, where the batch was larger than the rated capacity and poor mixing resulted. Lumps have occurred when the mixing time was shorter than the specified revolutions of the mixing drum. This latter is true especially in rich mixes. We have experienced lumps in concrete when the water contacts the cement before it is dispersed with the aggregate.

By periodic inspection of mixing equipment to insure efficiency; by using the equipment at the rated capacity and by charging the mixers so that all materials flow in together, we have virtually eliminated this trouble.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Tests of reinforced concrete slabs on elastic foundation

A. L. JOHANSSON, *Betong*, (Stockholm, Sweden) V. 32, No. 3, 1947, pp. 187-209
Reviewed by EIVIND HOGNESTAD

This paper reports investigations of reinforced concrete slabs on elastic foundations with a view toward airport runway applications. Tests have been made in the laboratory as well as in the field. A theory is advanced allowing the computation of ultimate loads. The agreement obtained between the experimental values and those computed according to the given formula is relatively satisfactory.

Das beton—A B C (concrete A B C)

A. HUMMEL, Berlin, 1948, 10th Edition, Published by W. Ernst & Sohn,
Retail price 15 RM plus postage
Reviewed by RUDOLPH FISCHL

Prof. A. Hummel of the Technical University in Berlin summarized in this book (first published in 1935) the latest findings in the field of concrete technology.

This 10th edition has been enlarged in response to the present situation in Germany. To save coal and energy, concrete is the preferred building material. Also discussed is the use of debris as aggregate, which will be a very important factor in the reconstruction of German cities.

Distribution of bulk cement

D. WERNER AND H. W. CARLSEN, *Cement och Betong*, (Malmö, Sweden) V. 23, No. 1 and 2, 1948,
pp. 17-27, 64-78
Reviewed by EIVIND HOGNESTAD

Swedish cement has generally been transported in sacks. Because of the present shortage of paper, however, transport by bulk is now promoted by the Swedish Cement Association. This review refers to three separate articles on that subject. Methods of transport by ship, railroad and trucks are discussed as adopted and adjusted to Swedish conditions. In the second article Mr. Carlsen calls attention to the fact that cement mixed with compressed air flows very easily. This property of cement is utilized for unloading of tank-trucks or railroad tank-cars. "Air-cushions" are also used to speed up the flow from cement bins. These cushions, situated at the bottom of the bin, consist of a porous plate through which compressed air enters the cement. The third article describes cement bins and simple but fairly reliable scale equipment for small jobs.

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Thermic drilling of concrete and stone

M. LEBRUN, *Structural Engineer* (London), V. 25, No. 2, 1947, pp. 57-76
HIGHWAY RESEARCH ABSTRACTS, Sept. 1948

As a result of researches by the Liquid Air Co. (Paris), it was found that heavy concrete work built by the Germans during the war could be readily broken up by an adaptation of the slice bar cutting process well known to furnace operators. The apparatus consists of a steel tube, sometimes filled with lengths of wire to equalize the flow, through which oxygen is injected after the extremity has been ignited. Details are given of the equipment for the thermic drilling of concrete and the conditions required for its use. Numerous examples of the demolition work carried out by this method are described, and suggested further possible applications of thermic drilling are listed.

New researches with concretes of rubble

A. STEOPOE, *Bulletin Inst., Natl. Cercetari Tehnol*,. V. 2, No. 1-4, 1947, pp. 233-242
CERAMIC ABSTRACTS, Sept. 1948 (M. Ha.)

Studies of concrete made with the rubble of demolished cities instead of natural materials from quarries showed the following results: (1) Rubble concretes settle about 20 percent more than other concretes due to the sharp edges of the rubble, which therefore contains more empty spaces than the natural materials with rounded grains. (2) If a part of the sand is replaced by equal parts of rubble and "brick" sand (from crushed brick), the resistance of the concrete obtained exceeds that of concrete made with quarry materials, especially after hardening periods of more than 28 days. (3) Drying shrinkage is greater in rubble concrete than in other concretes and it is accentuated by time; during the first 56 days of drying, shrinkage is almost proportional to time, while for quarry-material concrete the shrinkage varies in proportion to the logarithm of the time.

Belastungsglieder (load members)

A. KLEINLOGEL, Berlin. 1948, 7th Edition, Published by W. Ernst & Sohn,
Retail Price 7.80 RM plus postage — Reviewed by RUDOLPH FISCHL

This useful book by a well-known expert contains reactions, shear forces, moments and load members for 92 load cases for free supported beams as well as for beams with one or two fixed ends.

An introduction shows clearly the relations between shear forces, moments, load members which are identical with the cross-line distances in the fixed-point method, and gives the derivation of the Omega-numbers, which are used throughout the text to simplify the numerical work.

This publication covers practically all occurring load cases, every one shown with the shear and moment diagram, including construction lines for the curves. 32 tables allow a quick determination of the values in question.

Originally written as supplementary to the author's well-known formula books for rigid frames and continuous beams, this volume will be very useful also by itself for structural designers.

Nature and characteristics of hydraulic products manufactured in France

J. CLERET DE LANGAVANT—*Circulaire Serie F*, No. 38,
Institut Technique du Batiment et des Travaux Publics (Paris) — Reviewed by G. L. BLANCHARD

The object of this bulletin is to give the consumer of cement a clear and comprehensive picture of the characteristics of the numerous products presently on the postwar market

in France, from the practical standpoint of their expected performances and their utilization.

As the war and post-war conditions, particularly through acute shortage of coal, compelled the industry to meet the emergencies by the creation of new products, adapted as closely as possible to the wide variety of work involving the use of cement, the average customer is faced by the problem of choice among numerous materials of characteristics and properties only slightly different one from the other.

The object of this little pamphlet is to help the contractor to solve this problem, which he was not prepared to meet by his pre-war experience.

The current products are now thoroughly classified under official specifications similar to the A.S.T.M. regulations.

Water-tightness of concrete pipes, an investigation of testing methods

P. NYCANDER, *Betong*, (Stockholm, Sweden) V. 32, No. 3, 1947, pp. 210-246

Reviewed by EIVIND HOGNESTAD

The method generally applied in Sweden for hydrostatic testing of concrete pipes consists in setting the pipe upright and filling it with water after providing a watertight bottom of cement mortar. During the first 24 hours water is added to replace any loss. After the following 24 hours, the fall of the water level is measured; and this fall is taken as a measure of leakage in the pipe. In exceptional cases, measurements of the porosity of the concrete are made.

Laboratory tests are reported, the results of which show how the watertightness as measured by the method above is subject to large variations due to changes in air humidity, draft conditions and temperature. The influence of water pressure on leakage has been studied; and an empirical relationship is presented. An attempt is made to correlate porosity with leakage. The conclusion is advanced that hydrostatic tests after the Swedish method as well as tests based on measurement of porosity are very questionable. Attention is called to the hydrostatic test specified by the A.S.T.M. (C14-41). New tests are in progress attempting to adjust the latter test to suit Swedish requirements and practice.

Hydration and hydrolysis of the silicates and aluminates of calcium as a function of temperature. (Hydration et hydrolyse des silicates et des aluminates de calcium en fonction de la temperature)

M. J. BROCARD, *Annales* de L'Institut Technique du Batiment et des Travaux Publics, (Paris) No. 12 New Series, Feb. 1948

Reviewed by C. P. SIESS

The results of experiments to determine the action of water of different temperatures on the constituents of cement are presented. Curves and tables are given showing the concentration of calcium, silicon, and aluminum oxides as a function of time for tests run at temperatures of 15, 30, 50, 70, and 100 C. Curves are also given for the time to reach supersaturation versus temperature of the solution. The compounds studied included monocalcium aluminate, tricalcium aluminate, tetracalcium aluminoferrite, β-dicalcium silicate, tricalcium silicate, hydrated dicalcium aluminate, hydrated tetracalcium aluminate, cubic tricalcium aluminate and hydrated monocalcium silicate. The method used to prepare the solution of each of these compounds is described. The application of the results to the chemistry of cements is discussed.

Measurement of stresses in concrete by means of a photoelastic strain gage (Determination des contraintes dans le beton par la methode du tensometre photoelastique)

A. DE SOUSA COUTINHO, *Annales* de L'Institut Technique du Batiment et des Travaux Publics, (Paris) No. 20 New Series, May 1948

Reviewed by C. P. SIESS

An apparatus is described for measuring stresses in concrete by observing the polarization of light reflected from a back-silvered glass mirror embedded in the surface of the

concrete. To determine the direction and magnitude of the principal stresses in the mirror several observations must be made at each point, and readings must be taken near the edge of a small hole 2 to 3 mm. in diameter drilled in the center of the mirror which is itself about 20 mm. in diameter and 6 mm. thick. It is claimed on the basis of both theoretical and experimental studies that stresses in the mirror are produced only by stresses in the concrete and not by deformations alone. The development and verification of this method of measurement is described in detail. Applications in the laboratory and in the field are discussed, and some results presented.

New power station at Croydon

Concrete and Constructional Engineering, (London) V. 43, No. 6, June 1948, pp. 171-175 Reviewed by GLENN MURPHY

This article describes the plain and reinforced concrete construction involved in the 315,000 kw. power station at Croydon.

The concrete bases for the generators rest on a concrete slab reinforced with old rails. The bases for the columns of the power-house are plain concrete and the foundations for the 300-ft brick chimneys are reinforced with a double layer of radial mild steel bars.

Reinforced concrete construction is used for the 34-ft deep elevator pit.

Two sets of reinforced concrete filters, each approximately 25 ft wide and 33 ft long, are provided for each boiler. Electrostatic precipitators for the extraction of dust from the flue gasses are installed in the filters.

Nine cooling towers each over 250 ft high with a capacity of 3½ million gallons of water per hour are projected. Each tower will contain 60 diagonal reinforced concrete columns supporting the walls of the towers.

Several photographs are shown indicating the types of forms used and other details of construction.

Possibilities of improving the quality of concrete

VILIAM FIGUS, *Stavivo*, V. 26, No. 1, pp. 6-10, 1948 CERAMIC ABSTRACTS, Sept. 1948 (R. B.)

The quality of concrete may be improved by (1) condensing the concrete by means of vibration and pressing under high pressure with simultaneous heating (Freysinet's method), (2) squirting the concrete by means of compressed air (torket method), (3) condensing the concrete in its hardened state, (4) using vacuum concrete which has a strength 2 to 3 times greater, (5) using the centrifugal force resulting from rotation (for tubes, masts, etc.), (6) stamping the concrete, and (7) shaking it by vibration to eliminate the surplus water. Theoretically, it is possible to attain a maximum density of the concrete of 0.98, i.e., when the porosity is the lowest, 0.02 percent. It is therefore possible to improve the strength by reducing the coefficient of water surplus. Before 1906 Feret developed an equation for computing the strength of concrete approximately, $k = K\left(\frac{O_c}{1-O_s}\right)^2$, where O_c = the volume of the cement, O_s = the absolute volume of the admixtures contained in a unit of the volume of freshly made concrete, and K = a coefficient dependent on its age and the manner of storing. This coefficient amounts approximately to 2200. M. Roche, of the technical university of Zurich, has carried out about 10,000 tests to verify Feret's formula 5 diagrams.

Prestressed in situ concrete highway bridge

Surveyor and Municipal and County Engineer, (England) V. 107, No. 2942, June 25, HIGHWAY RESEARCH ABSTRACTS, Sept. 1948

Nunn's Bridge with its effective span of 74 ft and depth of 50 in., the first pre-stressed concrete cast in situ highway bridge to be built in England is an example of how modern methods of construction can solve an old problem. When the Witham Fourth District Drainage Board was faced with the need to replace the three-span brick-arched structure built by Rennie about 40 years ago, there were many difficulties. Summarized, the main factors to be considered in the new design were to cross the drain by a single-span bridge capable of carrying modern loads, to strengthen the end abutments, and at the same time keep constructional depth within reasonable limits.

It was decided to use pre-stressed concrete, as it was felt that it provided the ideal solution to these problems and would, in addition, reduce weight and keep the tonnage of steel as low as possible.

Following the removal of the old arch superstructure and the provision of new bearings supported on piles, steel joists, trussed to increase their stiffness, were temporarily supported on the old piers and a timber deck laid across them. The five main beams were cast on this deck. Each beam contained twelve cables, which were prevented from coming into actual contact with the concrete by means of sheaths. Each cable was of about 1-in. diameter, and consisted of twelve high-tensile steel wires over a bobbined wire core. When the specially high grade of concrete which was used in the beams, cross beams and deck had reached the specified strength of 6,000 psi, a tension of nearly 70 tons psi was applied to the cable by means of hydraulic jacks. By this means, what might be termed a reverse loading was introduced. The cables were later grouted under pressure.

An initial compression of nearly 2,000 psi was thus induced in the bottom flange of the beams with a compression of just over 100 psi in the deck. This meant that the application of the Ministry of Transport live loading to the bridge would just be sufficient to overcome the compression stresses in the bottom flanges.

Learnings and benefits gained from practical experience during the erection and later testing of reinforced concrete structures in Switzerland 1924-47

M. Ros, *Betong*, (Stockholm, Sweden) V. 32, No. 4, 1947, pp. 263-316 Reviewed by EIVIND HOGNESTAD

This article is translated into Swedish from a manuscript of a lecture given before the Swedish Concrete Assn. During the last 23 years tests have been made on about 80 reinforced concrete structures by the Swiss federal institute for testing of materials. These structures were very different in design and intended for various purposes. Their sites present wide differences in climatic conditions. A number of diagrams give the highlights of about 20 of these tests, the majority of which concern bridges. Through a discussion of tests, measurements and observations made in the field as well as in the laboratory Dr. Ros advances the conclusion that stresses and deformations within the limits of permissible stresses should be studied, as before, by means of the classical elastic theory. The plastic theory, however, applies to failures caused by static or repeated loads as well as buckling of reinforced concrete columns subject to central or eccentric loads. A second important point is made in discussing practical cases where the plasticity and creep of concrete is definitely beneficial. An empirical formula is presented relating modulus of elasticity and compressive strength of concrete. The author points out how knowledge of the average E, as computed from the elastic behavior of structures, allows sufficiently accurate determination of the ultimate compressive strength of concrete and thereby the effective factor of safety for the structure.

The present Swiss specifications for reinforced concrete structures, based on specified allowable stresses, make a division between primary and secondary loads. Dead and live load, centrifugal force, and impact are primary loads. Snow, wind, brake and friction forces as well as influences of temperature and creep are regarded as secondary loads. Experience from erected structures, intimately compared with knowledge gained in the laboratory, justifies an increase in allowable stresses for concrete and steel when primary and secondary loads both are considered. Furthermore, increasing stresses are allowed for three groups of structures in the following order: (1) slabs, T-beams and ribs under 5 in. thick; (2) rectangular cross-sections, beams and slabs, over 5 in. thick; (3) slabs supported on all edges, flat slabs and similar highly indeterminate structures over 5 in. thick.

In order to prevent corrosion of reinforcement, special care is taken to insure that cracks formed in reinforced concrete structures are as fine and uniformly distributed as possible. End hooks are indispensable to secure reliable anchorage for reinforcement bars. Practically watertight concrete made of high-grade aggregates, having the least possible water-content and a high unit weight (over 150 lb per cu ft), is practically frost resistant. Dr. Ros closes with a slogan rather characteristic of Swiss engineering: Quality matters more than quantity.

Substructure analysis and design

PAUL ANDERSON, Irwin Farnum Publishing Co., Chicago, 1948, $4.50 Reviewd by RALPH W. KLUGE

An interesting and well-written text-book on this subject. Its emphasis is on the analyses leading to the design of various foundation structures, although there are many illustrative problems dealing with their general design. The text also includes considerable general descriptive material on methods of construction and procedures. There are numerous references cited for those who wish to pursue further the topic under discussion and a number of tables giving useful data. Each chapter ends with the usual problems designed to test a knowledge of the preceding text.

The book begins with a discussion of earth pressure including a brief history and review of the classical earth pressure theories, applications of the elastic theory, and explanation of graphical methods for analyzing earth pressures. Steel, timber and reinforced concrete sheet piling are next discussed and a graphical analysis for determining moments in a sheet piling wall is explained. Considerable discussion is devoted to soils and their properties, bearing tests, allowable soil pressures and various methods of analyzing the bearing capacity of a soil for a given footing.

Following this is a chapter on single and combined footings of various shapes, raft foundations, grillage footings, and a brief discussion of underpinning. Steel, timber and concrete piles are next taken up with a discussion of their advantages and disadvantages, their spacing and driving. Several pile-driving formulas are discussed and results of their application compared in a numerical example. The next chapter is devoted to a rather extensive analysis of pile groups.

Subsequent chapters cover cofferdams, open and pneumatic caissons, break waters, wharves, bridge piers and abutments. An analysis of the forces acting upon each of these structures is given as well as descriptive material concerning types and structural arrangements.

The text concludes with five appendixes which include a table of dimensions and properties of various sheet piling sections; formulas for fixed-end moments and end reactions of beams subjected to hydrostatic loads; tables evaluating the trigonometric expressions involving the angles of internal friction, ϕ and slope δ, in the formula for active and passive earth pressures; and excerpts from the New York Building Code covering foundations and bearing capacity of soils.

Soil mechanics in engineering practice

KARL TERZAGHI AND RALPH B. BECK, First edition, 566 pp., illustrated, John Wiley & Sons, Inc., New York; Chapman and Hall, Ltd., London, 1948, $5.50 Reviewed by M. G. SPANGLER

Engineers of the English speaking world have long awaited and eagerly anticipated this volume by the "father of soil mechanics" and one of his very able pupils. Nor will the practicing engineer be disappointed when he turns to this book for guidance in meeting the problems which arise in the design and construction of structures which involve the soil. It contains a wealth of information based upon the extensive experience of the authors, information which is clearly presented and consisely stated. It is not a book for the beginner in the field nor is it suitable as a classroom text, except for the most advanced students.

The book is divided into three parts. Part A deals with the physical properties of soils and occupies about one-fourth of the volume. In this part, the index properties by which a soil may be identified and described, are presented. A chapter on hydraulic and mechanical properties gives basic information relative to permeability of soil and the application of the Darcy law. The authors' discussion of permeability and coefficient of permeability is very helpful in clearing up some of the confusion which exists among the various professional groups such as civil engineers, soil physicists, geologists, etc. relative to the meaning of various terms employed in connection with the flow of fluids through porous media. This chapter also includes discussions of effective and neutral stresses, seepage forces, shearing resistance and the compressibility and consolidation characteristics of soils, particularly clay soils.

A third chapter in part A is devoted to drainage of soils, including capillary phenomena and moisture movement, drainage by gravity and by electro-osmosis and a discussion of frost heave and its prevention. Although this chapter seems to be out of place in the part of the book which deals with the physical properties of soil, the material presented is timely and of great practical value.

Part B, which also occupies about one-fourth of the volume, consists of three chapters devoted to various phases of theoretical soil mechanics, such as states of plastic equilibrium, lateral earth pressures, soils bearing capacity, stability of earth slopes, settlement of foundations, seepage and piping. As a generalization, it may be said that part B of this book is a shortened version of the senior author's book "Theoretical Soil Mechanics," John Wiley and Sons, 1943.

Part C constitutes approximately one-half of the book and contains the meat and substance of the authors' contribution. The title of this part is "Problems of Design and Construction" and its contents are well described by this designation. Excellent discussions of the practical aspects of soil exploration, earth pressure and stability of slopes, foundations, and settlements of structures due to exceptional causes such as vibrations, lowering of the water table, and construction operations are embodied therein.

The practicing engineer will find much of value in this part of the book. Particularly he will be interested and helped by the discussion of various methods of soil exploration and the conditions under which each of the methods is appropriate. Practical information relative to the spacing and depth of bore holes is given, along with a discussion of the economics of soil exploration. The chapter on foundations is especially well done. In addition to the more usual type of discussion of spread foundations, rafts, piers, pile foundations and dam foundations, a very enlightening article on river bed scour is included in this chapter.

The authors recognize a number of topics of interest in the chapter of their book which belong in Part C, but which have not been included because of lack of space. An appendix has been added which provides the reader with comments and selected references

concerning these topics which include subgrade studies for highways and airports, the design of sheet-pile bulkheads, the design of cofferdams, earth tunneling, the design of culverts, and methods of grouting soils with cement and chemicals.

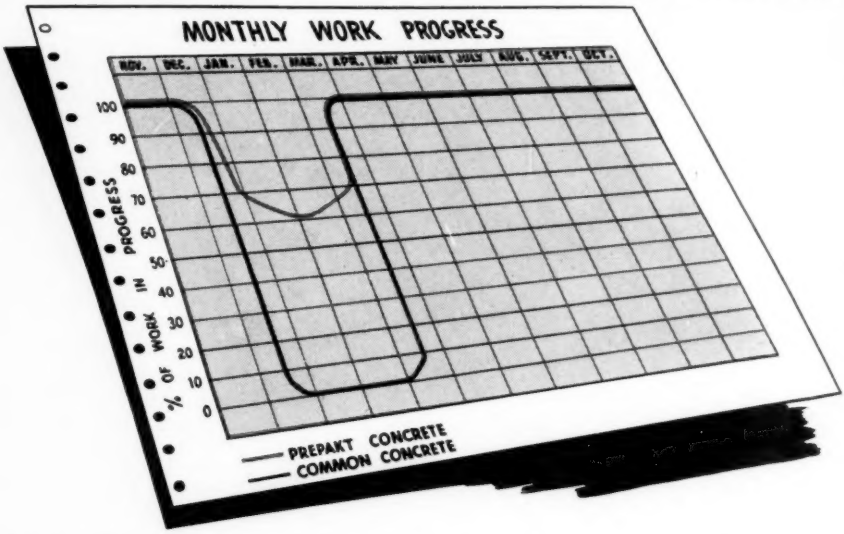
The book contains many line diagrams illustrating the thoughts and ideas expressed by the authors, but there are very few photographs of actual construction operations. It is written in a style which will appeal to the practitioner in the field and extensive mathematical discussions are conspicuous by their absence. In every way the book lives up to its stated purpose—for the successful application of soil mechanics to the design and construction of foundations, retaining walls and earth structures.

ACI News Letter

Vol. 20 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1948

PREPAKT REPORTS . . .

. . . ON EXTENSION OF THE CONSTRUCTION SEASON


MONTHLY WORK PROGRESS
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ACI

News Letter Contents

November 1948

On the cover—This view shows early construction progress in the building of a bridge link for Pittsburgh's Penn-Lincoln Parkway; here, steel centering is in place, pouring is virtually complete on the first rib of the arch, and the wooden forms are being erected on the second arch. The $1,800,000 structure will be known as the Commercial Avenue Bridge, and will carry the four-lane parkway from Edgewood across Nine-Mile Run to the eastern approach of the tunnel that will be built under a portion of the Squirrel Hill section of Pittsburgh.

Photograph by Blaw-Knox

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Board adopts report on ACI goals and staff to attain them

Recognizing the Institute's recent growth, present stature and means and opportunities for increasing both, the Board of Direction adopted at its recent fall meeting a report of its special committee setting up future ACI goals and the committee's broad recommendation for attaining them.

Members will be interested because a clear and important implication of the future plan is toward active participation in its affairs by a greater proportion of the membership through the facilities of a gradually enlarged ACI staff.

The Board committee was appointed by President Blanks in May 1948 with Douglas E. Parsons (past president and present chairman of the Technical Activities Committee) as chairman. Frank H. Jackson, vice president, and Stanton Walker, past president, completed the original committee membership. Mr. Blanks later appointed Henry L. Kennedy, Myron A. Swayze and Harry F. Thomson, directors, to serve with the three first appointed in the consideration and report of recommendations which were submitted to all Board members September 20 and were discussed and approved without dissent at a regularly scheduled meeting of the Board at which 14 of its 19 members were present.

These are the goals approved toward which ACI activities are to be directed:

"1. An increase in membership to approximately double the present membership in ten years and ultimately to include substantially all persons who would be usefully served by the Institute.

"2. Increase in the number of and improvement in the technical quality of papers with

particular emphasis on a larger number of comparatively short papers.

"3. Broaden the scope of technical papers to include more of those describing methods of construction, architectural features and details of techniques successful in obtaining attractive and durable structures.

"4. Increase in the quantity and scope of technical committee reports.

"5. Increase in the technical value and appeal of convention sessions.

"6. Increase in the number of and in the importance of discussions of technical papers and committee reports.

"7. Development of a policy of taking the Institute to the members, such as by increasing the number of regional meetings and the holding of local conferences."

The remainder of the report of the committee dealt with many elements of the work of the Institute and the staff to do that work. The following are excerpts from the committee report:

"The ten-year index was published in 1948 as the first step in the preparation and maintenance of an adequate index of the *Proceedings* to fill a long-recognized need. Notable improvements have been made in the appearance and legibility of the printed pages of the JOURNAL.

"The output of technical committees has not kept pace with the increase of membership. A few of the committees have prepared excellent reports but too many of them have been inactive. Their reorganizations last year and the appointment of new committees now being organized may result in greater constructive activity.

"Although convention sessions have been interesting and of value, their improvement seems possible. Despite a number of outstanding exceptions, convention papers tend to be so long and so filled with detail that listeners' attention is not held. Often too much is scheduled for single sessions and too little interesting oral discussion has been forthcoming. These faults are common to the more remote as well as to the recent past. Other technical organizations have not avoided them. Perhaps the greatest shortcoming during all periods with which the committee is familiar is the failure to enlist a larger number of its members in the activities of the ACI.

"On the whole the recent progress of the ACI has been gratifying. No one group is responsible for it. . . .

"Papers are being prepared for publication with reasonable promptness. Recent additions to the staff have made this possible also despite the substantial increase in the volume of publications. More papers are being edited by the staff than heretofore; extensive revisions are being made in some of them. The committee considers that the amount of staff aid to the authors should be increased and that the staff occasionally should prepare articles on subjects of wide interest when the technical facts are available and no other suitable authors can be found.

"Similar staff assistance is needed in the editing of technical committee reports. Indeed, some reports have not been published and others have been delayed many months because the original author needed but failed to receive sufficient assistance from a technical editor. In addition, the willing and skilled services of potentially valuable workers on technical committees are not being utilized because of their known inability to prepare well organized and readable reports. The Board and the TAC, rather than the staff, must assume primary responsibility for organizing new technical committees and for directing reorganizations when needed. It is the function of the staff to give administrative and technical assistance when needed, to prod tactfully, and to inform the TAC and the Board on committee activities. An increase in this aid seems desirable.

"The importance of the present and the desired future editorial functions of the ACI staff and the relations of the staff to the Committee activities are stressed because of their bearing on the present questions. The committee feels that the success with which the staff obtains the cooperation of volunteer workers on technical committees, of authors of papers and of reviewers of articles, and its skill and promptness in the editing and publishing of papers and reports is the principal measure of its adequacy.

"The committee feels that the present staff measures up well by these criteria. . . .

"After considering the objectives of the ACI and the functioning of the staff, your committee recommends the following for consideration by the Board:

1. Enlargement of staff:
   a) By a man, preferably with engineering training or experience, who is qualified to assist in editorial work, as already authorized by action of the Executive Committee.
   b) By an engineer qualified to coordinate, organize and expedite committee activities and to serve as a field secretary.
   c) By additional stenographic and clerical assistance as needed.
2. Increased scope of staff functions:
   a) By editing and rewriting papers of satisfactory content but unsatisfactory editorial treatment.
   b) By working closely with committees in organizing their work and preparing reports suitable for publication.
   c) By developing a program for bringing the activities of the Institute to the attention of a greater number by such field activities as formal regional meetings, informal group meetings, calls on members and prospective members, etc.
3. Increased activity of staff:
   a) On completing and maintaining the index of technical papers published in the *Proceedings*.
   b) On improving the Current Reviews section of the JOURNAL.

c) On efforts to acquaint prospective new members with services of the Institute by means of letters and literature mailed from Institute headquarters.

4. Increased space. Make immediate effort to secure sufficient space to accommodate present and prospective staff; necessity of going to another building should not be overlooked.

"The committee recognizes that it may not be feasible to employ both the editorial and the technical assistants immediately. In that event, the editorial assistant should come first. However, the committee wishes to emphasize that something must be done about additional office space, and that at once. The committee has considered the report made by Mr. H. F. Thomson as a result of his recent visit to the office and has also studied a floor plan of the present facilities. It is of the firm opinion that any staff, no matter how efficient, would be hindered by the crowded conditions. . . ."

Following Board discussion of an eighth goal to be added to the seven presented in the committee's report and not considered by the committee as within its original purview, Mr. Jackson moved and Mr. Parsons seconded a motion adopted without dissent that the existing ACI building fund committee be reappointed to continue its studies of the feasibility of an ACI-owned building for national headquarters.

New York convention program takes shape

Broad outlines of the 45th annual convention program in six sessions, February 23, 24 and 25 are now almost as final, except for details in two or three items, as they could well be as a working plan and intention.

General convention registration will begin 9 a.m. Wednesday, February 23, Hotel Pennsylvania, New York (by that time Hotel Pennsylvania, now Statler-managed, will become Hotel Statler—Statler-owned).

A considerable number of Institute technical committees have indicated their preliminary intention of meeting in the period provided—the entire forenoon of February 23. Provision will be made for adjourned sessions of the committees, insofar as possible, in the course of the evenings, because there will be no evening general sessions on any of the convention days.

It was felt by members of the New York ACI Convention Committee headed by M. J. McMillan, and by members of the Technical Activities Committee, that New York City offers many competing attractions. Members of the Institute are reasonably assured also that if they wish to avail themselves of New York opportunities for evening entertainment, the local committee will undertake to set up a plan to handle ticket reservations in advance of convention week.

2:00 p.m., Wednesday, February 23

The one general session the afternoon of February 23 will be divided into two parts; but they are inter-related parts and thus are joined in one program which seems to have large possibilites of interest in the area from which much of the convention audience will come.

B. D. Tallamy, now superintendent of the New York State Department of Public Works, formerly chief engineer, Division of Construction; and Charles M. Noble, chief engineer, New Jersey State Highway Department are scheduled as previously announced to present regional concrete paving problems of construction and maintenance as a basis for general discussion—of wide interest not only to

active road builders but to students of concrete generally, looking to solutions of problems of durability under severe exposures.

The second half of that session will have to do with air entrainment in concrete. J. F. Barbee, Ohio State Highway Department, already has in the hands of the Technical Activities Committee a paper reducing hundreds of pages of literature on the subject of air entrainment in concrete to what he believes to be the pertinent essentials of the subject so far as we have gone.

Under the same general heading Robert F. Blanks and W. A. Cordon, both of the Bureau of Reclamation, will present in digest a paper entitled "Practices, Experiences, and Tests with Air-Entraining Agents in Making Durable Concrete." All these papers are scheduled for post-convention publication.

9:00 a.m., February 24

This second general session will have to do with notable structural problems in the advancement of reinforced concrete.

C. P. Siess and Nathan M. Newmark will make a digest presentation, as a basis for discussion, of their paper under the title "Rational Analysis and Design of Two-Way Concrete Slabs," which will have publication in the JOURNAL in advance of the convention. Important contributions to discussion are anticipated.

Frank E. Richart's two-part paper, "Reinforced Concrete Wall and Column Footings," first part appearing in the October JOURNAL, second part in this issue, has important implications with reference to the future in design of reinforced concrete wall and column footings. The contribution is doubly important perhaps because the investigation reported is the first extensive study of reinforced concrete footings since the work of Dr. Talbot published in 1913.

Professor Richart will make a brief presentation of outstanding elements of his contribution as a basis for discussion.

Robert E. Glover, Bureau of Reclamation, Denver, is also preparing for the structural session a paper having to do with the use of various laboratory aids in the design of concrete structures. Particularly interesting are the interferometer polariscope, the Begg's deformeter, and the Stresscoat. These items, as used by the Bureau of Reclamation, are not items of research equipment, but are practically production line design tools.

12:30, an Institute luncheon—presentation of annual awards

In avoiding an evening session, the Technical Activities Committee and the New York ACI Convention Committee are in agreement that the one festive event of the convention should be a noon-day affair. Highlight of the occasion will be the annual presentation of awards by President Blanks.

February 24, afternoon session

Following the luncheon, after an interval to be announced later, there will be a short session starting with the report of tellers on the annual election, the induction of new officers and the address of retiring President Blanks.

One technical event will follow consisting of two papers, yet to have detailed announcement, under the general heading "Problems in the Production and Use of Ready Mixed Concrete," with considerable discussion anticipated.

February 25, two sessions

At 9:00 a.m. Friday, Prof. S. J. Chamberlin, chairman, and Prof. George W. Washa, secretary, will preside over the annual session of Committee 115—Research. This has been a session of increasing interest since its inauguration as an annual convention feature in 1937—except, of course, for two war years in which token conventions were held with brief business meetings only.

Continued on p. 8

The committee's annual questionnaire of many diverse agencies in the field of concrete research is in progress and a closely packed session will no doubt result, made up of very brief reports of research techniques and preliminary research findings.

At 2:00 p.m., the final session of the convention will feature two papers on "Cast-in-Place Architectural Concrete"—one on the design elements of such work by John J. Hogan and the other on construction features and details by E. B. Oberly—with A. J. Boase as co-author of each paper—all of the Portland Cement Association.

Ralph W. Kluge, formerly National Bureau of Standards, now of Purdue University, will present Bureau findings in reference to lightweight aggregate concrete—something that is commanding more and more interested attention in the field of concrete.

One more paper cannot yet be named specifically, but it will have to do with important developments and progress in the manufacture of concrete masonry units.

Who's Who This Month

A. C. Grafflin

As a comparatively new member of the Institute Mr. Grafflin appears as a JOURNAL author for the first time. "Cemenstone Precast Construction" which appears on p. 204 is a written version of information presented by Mr. Grafflin at the 44th annual convention in February, 1948. A native of Baltimore, Maryland, Mr. Grafflin attended the Johns Hopkins University and Maryland Institute, leaving to enlist in the Marine Corps in 1917. Upon return to the United States in 1919 he became associated with J. G. Valiant Co. as architectural designer and eventually became first vice president and member of the firm, with offices in Philadelphia, Baltimore, Washington and Paris. In 1936 he organized Grafflin Associates, Inc. and the Brookwood Development Co. in Baltimore for the development of properties and the building of fine homes. Early in 1942 he was commissioned in the Corps of Engineers and served overseas in the Aleutian Islands and in this country at the War College. Upon his separation from the army in 1945 he joined the Cemenstone Co. in Pittsburgh as vice president and general manager.

M. A. Craven

M. A. Craven makes his first contribution to the ACI JOURNAL this month in a paper entitled "Sand Grading Influence on Air Entrainment in Concrete". A member of the Institute since 1943, Mr. Craven is a native of Wellington, New Zealand who recently spent about 18 months in the United States and Canada studying American techniques and practices in the use and manufacture of concrete. His present paper was written while he was taking graduate work with Prof. Raymond E. Davis at the University of California in 1947. Mr. Craven is engineer for Certified Concrete, Ltd. of Wellington, New Zealand. In October of this year he accepted an offer to direct the activities of the newly formed New Zealand Portland Cement Association.

E. W. Scripture, F. B. Hornibrook and D. E. Bryant

are authors of "Influence of Size Grading of Sand on Air Entrainment" which appears on p. 217.

Edward W. Scripture, Jr. has been a member of the American Concrete Institute since 1931 and since that time has contributed frequently to the JOURNAL

as an author and discussor of JOURNAL papers. He attended Harvard University where he obtained his A. B. degree in 1920, M.A. in 1922 and Ph.D in 1924. During World War I he served overseas with the 26th Division. In the period 1923 to 1930 Dr. Scripture served as research chemist for a number of organizations and in 1930 became director of research of the Master Builders Co., Cleveland, Ohio. In 1942 he was appointed a captain in the Corps of Engineers in the U. S. Army and served most of the time until his separation in January 1946 in the Office of the Chief Engineer, E.T.O. He was separated from the army as a colonel, and returned to the Master Builders Co. as vice president in charge of research, the position he now holds.

F. B. Hornibrook appears for the second time this volume year as co-author of a JOURNAL paper . In September, the paper describing work at the National Bureau of Standards was entitled, "A Study of Alkali Aggregate Reactivity by Means of Mortar Bar Expansions". As a former director of research of the Master Builders Co. he contributes information to the present paper.

Long active in ACI technical committee work, he is well known to ACI members and little more need be said by way of introduction.

D. E. Bryant, as the third co-author, has had diversified experience as a chemist in portland cement manufacture. He attended the University of Des Moines, Des Moines, Iowa and completed a course in business management at the Capital City Commerical College of Des Moines. In 1933 he was briefly employed by the Chrysler Corp. on time study work in Detroit, Michigan. From 1934 to 1944 he was employed by the Green Bag Cement Co. of Pittsburgh as control chemist, physical tester, assistant chemist and research chemist. In 1941 and 1942 Mr. Bryant took additional work in chemistry by attending night school courses through The Pennsylvania State College. From 1944 to 1946 he was plant chemist with the Longhorn Portland Cement Co. of San Antonio, Texas. He has been senior research assistant in the research laboratories of the Master Builders Co. since 1946.

Paul Baumann

"Use of Prepacked Aggregate Concrete in Major Dam Construction" is the first JOURNAL paper of Paul Baumann, Assistant Chief Engineer, Los Angeles County Flood Control District. Although a comparatively new member of the Institute Mr. Baumann has had long experience in engineering. A native of Bern, Switzerland he received his diploma in civil engineering from the Swiss Federal Institute of Technology, Zurich in 1918. He served as an officer in the Swiss Corps of Engineers during World War I and subsequently was engaged in field surveys and preliminary design of hydroelectric power developments in the Bernese Alps (in the vicinity of the dam described in his current paper). In 1920 he came to America as a member of a Swiss commission for economic and technical studies. While in the United States he resigned from the commission to take a job with the Fargo Engineering Co. of Jackson, Michigan. From 1921 to the present Mr. Baumann has been concerned with surveys, studies, design and construction of steam and hydroelectric power plants, irrigation and flood control projects. Since 1934 he has been assistant chief engineer of the Los Angeles County Flood Control District in charge of the dams and conservation branch. He has had numerous papers published in domestic and foreign technical periodicals and is active in the A.S.C.E. as chairman of its Committee on Hydraulic Data and Facts. Two of his papers have been awarded the James Laurie Prize and the Thomas Fitch Rowland Prize, respectively, of the A.S.C.E. His civic activities include membership in the Natural Resources Committee, California State Chamber of Commerce.

People, Projects and Plaudits

Franklin R. McMillan

Franklin R. McMillan retires this month after 24 years with the Portland Cement Association. He has been an Institute member since 1916 — an Honorary Member since 1943. Few men have served the cause of good concrete as has he.

His retirement as director of research of the Portland Cement Association was

F. R. McMillan

effective November 1, 1947 and was to have been complete retirement. But he was asked to stay on another year in a new position—assistant to the vice president in charge of research and development, to bring nearer to fruition projects he had directed for so long. He will continue to serve the association in a consulting capacity.

In 1928 Mr. McMillan gave ACI his "Concrete Primer" (V. 24, p. 495) which won the Wason Medal of that year and is with little doubt the Institute's most widely published work; more than 70,000 copies have been printed by ACI plus translations published abroad by ACI permission in at least a half dozen languages.

Mr. McMillan became president of the Institute in February 1936, serving one year as had become the pattern two years previous to his tenure. He had first become a member of the Board of Direction in 1930 as director from the former fifth district, serving in that position until his election as vice president in 1934. Following his term as president he served as a past president member of the Board until 1941.

From 1929 to 1940 he was active in the Advisory Committee, Publications Committee and the Program Committee whose responsibilities and functions were recently consolidated in the Technical Activities Committee. He had resourcefulness through wide acquaintance and judgment in fitting tasks to those who could and would do them. It was thus he became a particularly potent force as chairman of the Program Committee.

The last year has seen the publication of the first papers reporting "Long-Time Study of Cement Performance in Concrete" (see ACI *Proc.* V. 19) with which he has been long associated.

Mr. McMillan received his civil engineering degree from the University of Minnesota in 1905. From that year until 1909 he was in railway engineering construction, building construction, and irrigation surveys. First as instructor in 1909, then as assistant professor, he served the college of engineering of his alma mater especially in his aroused interest in concrete and reinforced concrete research.

In 1918 he was called to take charge of the concrete ship tests for the Emergency Fleet Corporation, U. S. Shipping Board. In the course of this work he developed the strainagraph for measurement of hull stresses which was used for the first

recording strain gage readings ever made on ships at sea. After the war he became associate structural engineer with the Turner Construction Co.; later a connection with Adolph Meyer, hydraulic engineer, Minneapolis in consulting practice.

Mr. McMillan joined P.C.A., in 1924. In 1926 became manager of the Structural and Technical Bureau, and in the following year became the Portland Cement Association's director of research.

A list of Mr. McMillan's major contributions to the literature of concrete is much too long to publish here. A partial list which has since grown much longer was published in the News Letter section, ACI JOURNAL, March-April 1937.

His committee activities have been extensive, not only in the American Concrete Institute but in the American Society of Civil Engineers and the American Society for Testing Materials. From 1925 to 1928 he was chairman of Committee E-1—Reinforced Concrete Building Design and Specifications (the old building code committee).

An outstanding work was in his connection with the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, beginning with the second Joint Committee in 1921 and continuing this work as secretary of the third Joint Committee which reported in 1940.

So far as is known, his major hobby is work. Photography was developed as an aid to that work, prints from his Leica camera reinforcing the record in his always present notebook.

Acting President Pearson named Mr. McMillan as a representative of the American Concrete Institute at the Second International Congress for Bridge and Structural Engineering in 1936. With that occasion for overseas study he coupled an automobile inspection trip in Europe and filled eight fat notebooks with his observations in a ten-week period. Of what he observed he had 2,000 pictures as supporting evidence. Following the trip his "Some Comparisons of European and American Concrete Practice" was published in the March-April ACI JOURNAL 1937.

Arthur A. Levison

Arthur A. Levison, member of the American Concrete Institute since 1924 and increasingly well known in the field of concrete construction equipment, as vice president in charge of the Construction Equipment Department, Blaw-Knox Co., Pittsburgh, was recently elected a director, three-year term, of the Construction Industries Association, Inc. The organization is affiliated with the American Road Builders Association in which it constitutes the manufacturers division.

Mr. Levison was recently appointed to membership on ACI Committee 609, Vibration of Concrete, under the chairmanship of Harmon S. Meissner. He is also a new member of ACI Committee 621, Aggregate Selection, Preparation, Handling and Use, under the chairmanship of I. L. Tyler.

J. R. Tonry

J. R. Tonry, formerly chemist at the Martinsburg, West Virginia plant of Standard Lime and Stone Company, Baltimore, Maryland, has been appointed chief chemist at the Portland, Colorado plant of Ideal Cement Company, Denver, Colorado. A graduate of the University of West Virginia, Mr. Tonry has been in the cement business for the past ten years. He joined the Institute in 1947.

Ralph W. Kluge

Purdue University has recently announced that Ralph W. Kluge, a member of the Institute since 1938, has joined the staff of the school of engineering and engineering mechanics as associate professor of structural engineering, and will have immediate charge of instruction and research in reinforced concrete. Professor Kluge is a graduate of the University of Illinois, class of 1928, with a degree of B.S. in C.E.; in 1930 he received his M.S. in C.E. at the same university. From 1930 to 1941 he was special research assistant at the University of Illinois Engineering Experiment Station. From 1941 to 1948 he was a structural engineer engaged in research at the National Bureau of Standards. In addition to his ACI membership, he is a member of A.S.C.E. and Sigma Xi.

George A. Mansfield

George A. Mansfield has resigned his position as technical service director and division sales manager of the Huron Portland Cement Company, Detroit, Michigan, to become associated with William Moors Concrete Products, Fraser, Michigan. Mr. Mansfield advises that this change was made following the advice of his physician to eliminate extensive traveling. He has been Huron Portland Cement Co. corporation representative in ACI since 1944. Mr. J. A. Kaner is the new representative.

Thomas C. Kavanagh

A correction is in order for the item on p. 7 of the September News Letter which announced that Thomas C. Kavanagh had been promoted to the rank of assistant professor in the aeronautical engineering department of New York University. Actually, he resigned from the staff of N. Y. U. last June to accept a position as associate professor of civil engineering at The Pennsylvania State College where he is teaching in the field of advanced structures.

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New Members

Of 53 applications for membership received in September and approved by the Board of Direction, 30 were Individual, 6 Corporation, 12 Junior, 5 Student.

The geographical listing shows Massachusetts in front for the month with 10 new applicants. California and New York each provided 4 new applicants.

Our membership total October 1, 1948 became 4288.

Alabama

Coleman, Robert B., Jr., (Indiv.) Southern Research Institute, 917 S. 20th St., Birmingham 5, Ala.

Arkansas

Flowers, Elvis B., (Indiv.) 14 Crescent Drive, Mountain Home, Ark.

Schicker, Edward B. Jr., (Indiv.) Mountain Home, Ark.

California

Anderson, James, (Indiv.) Holmes & Narver, Okinawa Engr. Dist., APO 331, c/o P. M., San Francisco, Calif.

Bronsert, Harvey Wilfred, (St.) Box 146 La Sierra Station, Arlington, Calif.

Cartier, Felix W., (Indiv.) 870 N. Eucalyptus Ave., Hawthorne, Calif.

Thomas, Harry E., (Indiv.) P. G. & E. Laboratory, 4245 Hollis St., Emeryville 8, Calif.

Illinois

Arms, Leo M., (Indiv.) 33 W. Grand Ave., Chicago 10, Ill.

Symons, John G., (Indiv.) Symons Clamp & Mfg. Co., 4249 Diversey Ave., Chicago 39, Ill.

Viest, Ivan M., (St.) 105 Talbot Laboratory, University of Illinois, Urbana, Ill.

Indiana

De Rienzo, Harold, (Indiv.) 210 Dixieway South, South Bend 17, Ind.

Moore, Wm. S., (Indiv.) 401-402 St. Joseph Bank Bldg., South Bend 1, Ind.

Iowa

Parish, William L., (Indiv.) 1001 Kahl Bldg., Davenport, Ia.

Kentucky

Hill, Lee Robinson, (Jr.) 315 Guthrie St., Louisville, Ky.

Louisiana

Shilstone, Cecil Maxwell, (Indiv.) Shilstone Testing Laboratory, Inc., 510 Gravier St., New Orleans 12, La.

Maryland

Bornefeld, Charles F., (Indiv.) 1010 N. Charles St., Baltimore 1, Md.

Massachusetts

Corso, Jose M., (Jr.) 106 Hammond St., Cambridge, Mass.

Davis, Carleton Warner, (Jr.) 206 Pleasant View Ave., Braintree 84, Mass.

Dembo, Michael, (Jr.) 36 St. Stephens St., Boston, Mass.

Francolino, Charles A., (Jr.) 25 Lincoln Terrace, Winthrop Mass.

Graves, Alfred Green, (Jr.) 96 Pierce Road, Watertown, Mass.

Hart, Edward Conrad, (Jr.) 15 Tenney St., Cambridge, Mass.

Highsmith, James Albert Jr., (Jr.) 182 Oakland St., Medway, Mass.

Kenney, Howard French, Jr., (Jr.) 31 Holton St., West Medford, Mass.

McNiven, Hugh Donald, (Jr.) 89 St. James Ave., Boston 16, Mass.

Skarlatos, Mikhail Nicholas S., (Jr.) 50 Mt. Vernon St., Cambridge 40, Mass.

Montana

Lowe, William E., (Indiv.) 742B Fairhaven, Great Falls, Mont.

New York

Flay, George F., Jr., (Indiv.) 2 Park Ave., New York 16, N. Y.

Gabey, Martin A., (Jr.) 2233 Norton Ave., New York 61, N. Y.

Hans, Alexander A., (Indiv.) Weir Lane, Locust Valley, L. I., N. Y.

Hawley, Harry H., (Indiv.) Union College, Schenectady, N. Y.

Oklahoma

Spicers Construction Co., (Corp.) 7701 North Western, P. O. Box 1532, Oklahoma City, Okla. (L. C. Moore)

Oregon

Burgoyne, Carlyle F., (Indiv.) 1509 W. Fifth Ave., Eugene, Ore.

Pennsylvania

Minnick, Leonard John, (Indiv.) 306 Boyer Road, Cheltenham, Pa.

Tennessee

Dutton, Benson L., (Indiv.) Director, Div. of Engineering, Tennessee A & I State College, Nashville, Tenn.

Jones, Henry Chapman, (Indiv.) P. O. Box 641, Chattanooga, Tenn.

Texas

Leith, James B., (Indiv.) Box 111, Marlin, Texas

Kelley, J. Thomas, (Indiv.) 1506 Lawrence St., Pasadena, Texas

Roessner, R. Gommel, (Indiv.) 920 E. 37th St., Austin, Texas

Utah

Esquivel, Joaquin Y., (St.) University of Utah, Salt Lake City, Utah

Argentina

"Hormigon Elastico", Review, (Corp.) Lavalle 1171, 5°-piso, Buenos Aires, Argentina (Ivo Ghetaldi)

Weyer & Cia SRL, (Corp.) Belgrano 835. Buenos Aires, Argentina (M. Weyer)

"Zofra" S. A. (Corp.) Calle Lavalle 1171, Buenos Aires, Argentina (Zorislav Franjetich)

Australia

Watson, Frank Enos, (Indiv.) 16, Ford St., West Midland, Western Australia

Canada

H. G. Acres & Co., (Corp.) 2135 Culp St., Niagara Falls, Ont. (A. W. F. McQueen)

Gale, M. L., (Indiv.) 3176 W. 34th Ave., Vancouver, B. C., Canada

England

Messrs. John Laing & Son Ltd., (Corp.) Bunns Lane, Mill Hill, London N W 7, England (John McNeal Fisher)

Marcel, O., (Indiv.) 379, Chiswick High Road, London W. 4, England

Guatemala

Ochoa U., Francisco J., (St.) Callejon de la Cruz No. 6, Guatemala City, Guatemala, C. A.

Toledo, Jose Lopez, (St.) 3a C. P. No. 14, Guatemala City, Guatemala, C. A.

India

Mohile, N. H., (Indiv.) The Concrete Association of India, 1-2, Queen's Road, Fort, Bombay, India

Sweden

Eklof, Bertil Karl Emanuel, (Indiv.) Harspranget, Sweden

Switzerland

Frey, Roger, (Indiv.) Beethovenstr. 47, Zurich, Switzerland

N.S.G.A. and N.R.M.C.A. short course

The National Sand and Gravel Association and the National Ready Mixed Concrete Association have announced plans for their annual short course of instruction for technicians in the sand and gravel and ready mixed concrete industries at the University of Maryland, November 15 to 20, 1948. As in the past, the course is being sponsored by the College of Engineering of the University of Maryland and the two associations. The course will include: basic factors affecting quality of concrete mixtures; air-entraining concrete; fundamentals of design of concrete mixtures; laboratory demonstrations and discussions of tests of concrete; laboratory demonstrations and discussions of tests of aggregates; aggregate specification problems; and bituminous mixtures.

A.S.T.M. announces meeting dates

The American Society for Testing Materials has announced the following dates for their coming meetings: 1949 (fifty-second) annual meeting, Chalfonte-Haddon Hall, Atlantic City, N. J., June 27 to July 1, inclusive; 1949 spring meeting and A.S.T.M. committee week, Hotel Edgewater Beach, Chicago, Ill., February 28 to March 4, inclusive; 1949 west coast meeting, Fairmont Hotel, San Francisco, California, October 10 to 14, inclusive.

Honor Roll

February 1 to September 30, 1948

Prof. Newlin D. Morgan, Jr., University of Wyoming, continues to top our current Honor Roll with credit for 30 new Members for the period ending September 30, 1948. Professor Morgan's father, Newlin D. Morgan remains in second place with 15 credits.

Newlin D. Morgan, Jr. (Wyo.).....30
Newlin D. Morgan (Ill.).....15
Raymond E. Davis (Calif.).....14½
Thomas E. Stanton (Calif.).....14
Howard Simpson (Mass.).....10
James A. McCarthy (Ind.)..... 6½
Raphael Callejas H. (Guatemala).....6
Harrison F. Gonnerman (Ill.).....6
Henry L. Kennedy (Mass.).....6
Oliver G. Julian (Mass.)..... 5
Wm. H. Thoman (Colo.).....4½
E. I. Fiesenheiser (Ill.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Leon Venegas (Costa Rica).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
G. E. Troxell (Calif.).....3½
Robert W. Freeman (Calif.).....3
Ray C. Giddings (Calif.).....3
George E. Large (Ohio).....3
A. E. Mackney (Canada).....3
F. N. Menefee (Mich.).....3
Robert Morris (Colo.).....3
James M. Polatty (Ga.).....3
Richard A. Roberts (Calif.).....3
Joseph J. Waddell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
Frank E. Richart (Ill.).....2½
John A. Ruhling (Ill.).....2½
John C. Sprague (Ga.).....2½
Hugh Barnes (Calif.).....2
F. Campus (Belgium).....2
Harold S. Carter (Utah).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
H. J. Gilkey (Iowa).....2
Ernst Gruenwald (N. Y.).....2
Myle J. Holley, Jr. (Mass.).....2
Edward L. Howard, Jr. (Calif.).....2
R. R. Kaufman (Ohio).....2
John C. King (Ohio).....2
William E. Lumb (Canada).....2
Rene S. Pulido y Morales (Cuba).....2
Hugh E. Odor (Wash.).....2
George L. Otterson (Ark.).....2
Walter H. Price (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
C. H. Scholer (Kans.).....2
E. Copeland Snelgrove (England).....2
H. D. Sullivan (Wash.).....2
J. Neil Thompson (Texas).....2
E. W. Thorson (Colo.).....2
Juan Agustin Valle (Argentina).....2
Stanton Walker (D. C.).....2
Jose Luis Capacete (Puerto Rico).....1½
A. R. Collins (England).....1½
Milton Fromer (Calif.).....1½
Thomas C. Kavanagh (Pa.).....1½
Francis R. McCabe (Ore.).....1½
Eugene Mirabelli (Mass.).....1½
W. E. Parker (Canada).....1½
Dean Peabody, Jr. (Mass.).....1½
Jerome M. Raphael (Colo.).....1½
Clarence Rawhouser (Colo.).....1½
Anton Rydland (Calif.).....1½
Edward W. Scripture, Jr. (Ohio).....1½
Roy T. Sessums (La.).....1½
Harry F. Thomson (Mo.).....1½
John Tucker, Jr. (D. C.).....1½
Calvin T. Watts (La.).....1½
Harold E. Wessman (Wash.).....1½
Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
A. Amirikian.....1
Boyd G. Anderson.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1
F. Barona.....1
Frank H. Beinhauer.....1
Rene L. Bertin.....1
Robert F. Blanks.....1
A. J. Boase.....1
E. K. Borchard.....1
H. L. Bowman.....1
H. E. Burr.....1
John Butkus.....1
A. Carbone.....1
A. N. Dutta Choudhury.....1
C. O. Christy.....1
Miles N. Clair.....1
Rodger B. Collons.....1
R. Torres Colondres.....1
Jack E. Counts.....1
Walter Dardel.....1

J. Slater Davidson, Jr. 1
Harmer E. Davis 1
George C. Ernst 1
A. Finlayson 1
T. H. Fleming 1
R. E. Fowle 1
Zorislav Franjetich 1
Frank M. Fucik 1
Carmelo Galindez 1
Anthony J. Giardina 1
R. R. Gore 1
Emil A. Gramstorff 1
Eugene Guillard 1
George E. Hatch 1
F. E. Henman 1
Samuel Hobbs 1
W. C. Huntington 1
Phil C. Huntly 1
Wm. R. Johnson 1
H. D. Jolley 1
Edward F. Keniston 1
P. J. Kennedy, Jr. 1
Orla A. Larsen 1
B. Leon 1
W. G. Limbach 1
L. D. Long 1
Charles Macklin 1
John V. Maescher 1
Clyde P. Mason 1
F. T. Mavis 1
William G. McFarland 1
Douglas McHenry 1
F. R. McMillan 1
Carl A. Menzel 1
L. J. Meszaros 1
Oliver H. Millikan 1
Leonard J. Mitchell 1
Robert B. B. Moorman 1
John J. Mullen 1
R. P. Mumford 1
T. D. Mylrea 1
Amado B. Navoa 1
William T. Neelands 1
C H. Neill 1
N. M. Newmark 1
D. G. Norman 1
Wm. D. Nowlin 1
R. A. Plumb 1
James J. Pollard 1
Horace A. Pratt 1
Abdul Qayum 1
R. D. Rader 1
Thomas J. Reading 1
J. W. Rivett 1
H. H. Roberts 1
Fernando A. Rojas B. 1
Georges Ronai 1
E. Rosecrans 1
H. C. Ross 1
E. Gonzalez Rubio 1
C. E. Sandstedt 1
Edgar A. Sealey 1
C. Seshavatharam 1
Thomas C. Shedd 1
James M. Shivley 1
Joseph Sill 1
Francis P. Sloan 1
Gustave Spirz 1
Hale Sutherland 1
E. O. Sweetser 1
T. Thorvaldson 1
Dean P. Tsagaris 1
Walter C. Voss 1
Jukka E. Vuroinen 1
W. F. Way 1
Paul Weidlinger 1
J. M. Wells 1
C. D. Williams 1
H. Roger Williams 1
Merle D Wilson 1

The following credits are, in each instance "50-50" with another member.

Robert W. Abbett	H. N. Howe
George C. Alden	Walter L. Huber
Anderson, A. A.	J. W. Hubler
Oscar Vega Arguelles	Manuel Castro Huerta
A. Allan Bates	C. L. N. Iyengar
E. W. Bauman	Orville E. Jack
C. T. Bishop	Frank H. Jackson
Petrus Manzano Bonilla	M. E. James
	Bruce Jameyson
P. G. Bowie	E. B. Johansen
Harold W. Brewer	Harry D. Jumper
A. R. Brickler	J. W. Kelly
Boyd S. Brooks	Thomas M. Kelly
L. Rees Brooks	John J. Kennedy Jr.
Sterling Lowe Bugg	K. J. Krawczyk
B. W. Burns	H. M. Larmour
Jose Luis Casas	Robert E. Lee
Thomas F. Chace	L. G. Lenhardt
Wilbur H. Chamberlin	John F. Long
Richard de Charms	Wallace A. Marsh
R. V. Chate	Hubert F. McDonell
J. E. Christiansen	Callix E. Miller, Jr.
James H. H. Chun	Hugh Montgomery
H. W. Chutter	Ray M. Moorhead
Emil W. Colli	A. E. Myers
F. Coppen	Israel Narrow
Rolland Cravens	Elmer H. Oechsle
J. M. Crom	Francisco Santos Oliva
R. A. Crysler	Chester B. Palmer
Philip Davis	G. S. Paxson
Charles W. De Groff	Donald O. Peck
John G. Dempsey	Ben Poisner
J. B. Dobsen	Chesley J. Posey
J. E. Dunn	Emil H. Praeger
J. Dvorak	Herman G. Protze
John N. Eckle	Warren Raeder
Harlan H. Edwards	Dwight F. Roberts
A. C. Eichenlaub	Bernard L. Robinson
F. L. Ehasz	Fredric Rusche
Arthur D. Engle	Fred H. Ryan
William E. Feidt	Fernando Sanchez
Paul Fischer	Herman Schorer
Rudolph Fischl	Joseph J. Shideler
E. M. Fleming	Charles A. Shirk
Frederick E. W. Fowler	Chester P. Siess
Jaichand Hirachand Gandhi	R. Simons
	J. Singleton-Green
Leo V. Garrity	Charles M. Spofford
Carl V. Gezelius	A. R. Stuckey
Luis Alberto Truque Gurdian	M. O. Sylliassen
	Ruth D. Terzaghi
J. R. G. Hanlon	A. G. Timms
Bernard Harrison	Donald R. Warren
Frank W. Haws	J. C. Watt
H. L. Henson	J. D. Whittaker
A. E. Hiscox	William H. Wisniski
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An amendment to the By-Laws proposed

For the purpose of making the By-Laws consistent in Article II, Sec. 13 with the amendment of Sec. 7 already ratified, we the undersigned in accordance with the provisions of Sec. 1, Article VI—Amendments petition that Art. II, Sec. 13 be amended in the second paragraph to provide:

"The Vice Presidents, each in the order of his seniority in that office, shall discharge the duties of the President, in his absence."

M. A. Swayze
Paul W. Norton
Henry L. Kennedy
Charles H. Scholer
Harry F. Thomson
Hugh P. Bigler
R. B. Young
D. E. Parsons
F. H. Jackson
A. T. Goldbeck
H. J. Gilkey
Stanton Walker
Harvey Whipple
R. F. Blanks
H. F. Gonnerman

The foregoing petition was approved by the Board of Direction at its meeting October 7. Thus the proposed amendment will go on the ballot along with the candidates for office for terms expiring at the 45th annual convention in New York City, February 24.

Statement of the Ownership, Management, Circulation, Etc., Required by the Act of Congress of August 24, 1912, as Amended by the Acts of March 3, 1933 and July 2, 1946 of JOURNAL OF THE AMERICAN CONCRETE INSTITUTE published 10 issues a year at Detroit, Michigan for September, 1948.

STATE OF MICHIGAN... } ss
COUNTY OF WAYNE.... }

Before me, a Notary Public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily, weekly, semiweekly or triweekly newspaper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912, as amended by the Acts of March 3, 1933, and July 2, 1946 (section 537, Postal Laws and Regulations), printed on the reverse of this form, to wit:

1. That the names and addresses of the publisher, editor, managing editor, and business managers are:

Publisher, American Concrete Institute, 717 New Center Bldg., Detroit 2, Mich.

Editor, Harvey Whipple, 717 New Center Bldg., Detroit 2, Mich.

Managing Editor, None.

Business Manager, None.

2. That the owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding one per cent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a firm, company, or other unincorporated concern, its name and address as well as those of each individual member, must be given.)

American Concrete Institute, 717 New Center Bldg., Detroit 2, Mich.

Robert F. Blanks, President, c/o U. S. Bureau of Reclamation, Denver 2, Colo.

Herbert J. Gilkey, Vice President, Iowa State College, Ames, Ia.

Frank H. Jackson, Vice President, U. S. Public Roads Administration, Washington, D. C.

3. That the known bondholders, mortgagees, and other security holders owning or holding 1 percent or more of total amount of bonds, mortgages, or other securities are: (If there are none, so state.)

NONE.

4. That the two paragraphs next above, giving the names of the owners, stockholders, and security holders, if any, contain not only the list of stockholders and security holders as they appear upon the books of the company but also, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduciary relation, the name of the person or corporation for whom such trustee is acting, is given; also that the said two paragraphs contain statements embracing affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner; and this affiant has no reason to believe that any other person, association, or corporation has any interest direct or indirect in the said stock, bonds, or other securities than as so stated by him.

5. That the average number of copies of each issue of this publication sold or distributed through the mails or otherwise, to paid subscribers during the twelve months preceding the date shown above is.....(This information is required from daily, weekly, semiweekly, and triweekly newspapers only.)

HARVEY WHIPPLE,
(Signature of editor)

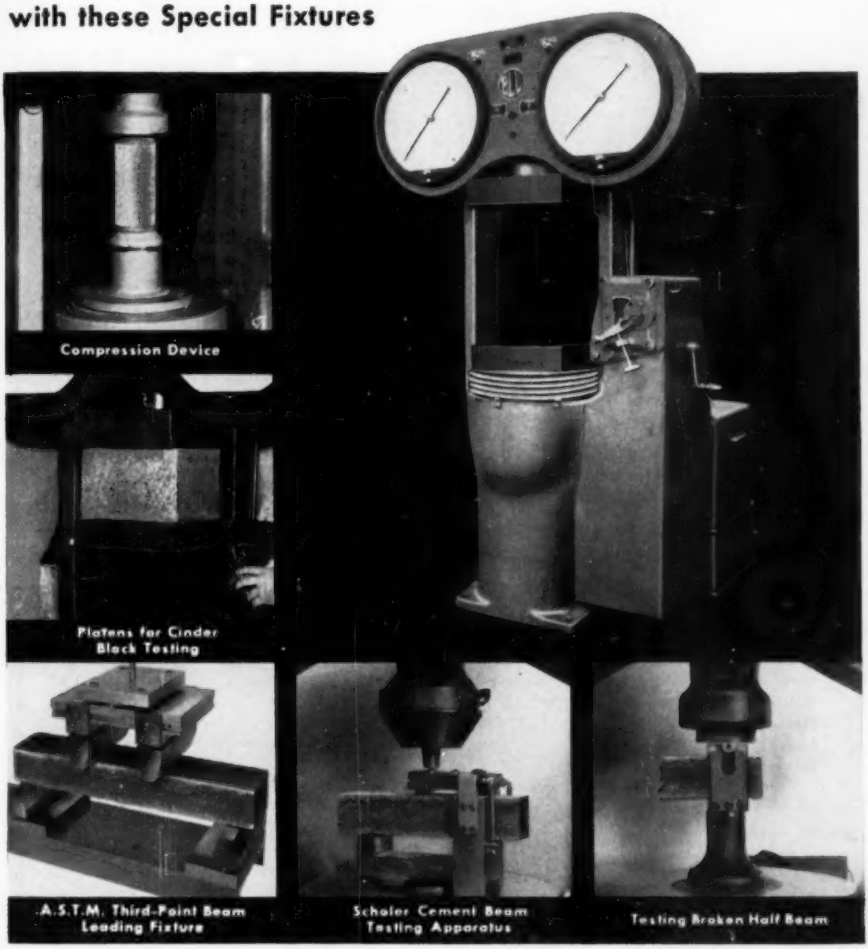
Sworn to and subscribed before me this 20th day of September, 1948.

JEAN WILLIAMS, Notary Public
(My commission expires Nov. 19, 1948)

[SEAL]

new fields of use for the new BALDWIN Cement Testing Machine

with these Special Fixtures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Platens for Cinder Block Testing
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Testing Broken Half Beam

The broadened opportunities for cement and concrete testing opened by the new Baldwin 300,000 lb. machine are further expanded by available special fixtures.

These devices provide more detailed information, and facilitate the direct determination of such data as crushing strength, beam characteristics, and three-point loading reactions. Results provide valuable design factors which cannot always be inferred from straight compression tests.

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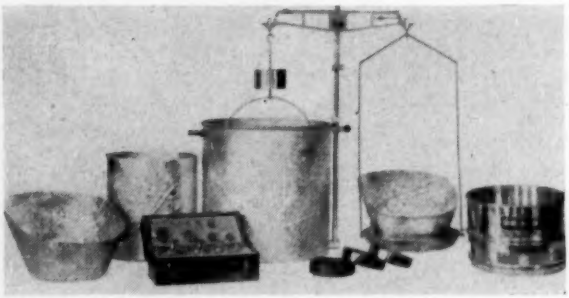

Designed for use in performing the following tests:

1. **Specific Gravity of Fine and Coarse Aggregates.**
2. **Free Moisture or Absorption of Fine and Coarse Aggregates.**
3. **Silt Determination.**
4. **Analysis of the Constituents of Fresh Concrete.***

These tests are performed with one article of equipment without recourse to heating or drying any of the materials. (All that is needed is the apparatus and fresh water.)

The apparatus is rugged and permanent, portable, and the results obtained are within a degree of accuracy required by the most rigid specifications. All tests can be performed at a minimum of time.

In addition to the equipment illustrated, a large container with cover is furnished. This container is used for washing the material when tests are made in the field, and when not in use all the equipment is packed into this container, making it a compact field kit.

Also a short stirrup and pan is furnished for use in dry weighing for making sieve analysis of aggregates for concrete.

The pail with handle (overflow bucket) shown in illustration has a capacity of ¼ cu. ft. and may be utilized as a measure for unit weight determinations of aggregates.

Furnished with "Manual of Control Tests for Portland Cement Concrete" serving as directions for use.

*(See Journal of the American Concrete Institute, Vol. 1, December, 1929, page 202; also A. S. T. M. 1931 Proceedings, Vol. 31, Part I, page 383, and Iowa State College Bulletin 113, "A Proposed System for the Analysis and Field Control of Fresh Concrete," Vol. XXXI, May, 1933, No. 49.)

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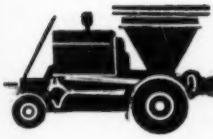

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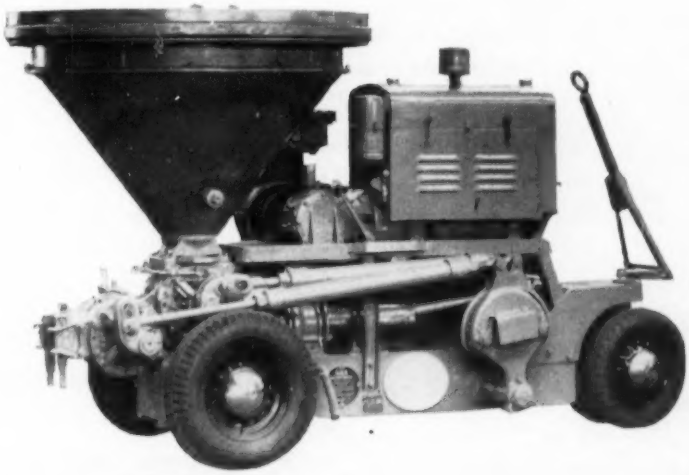

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San Rafael, California

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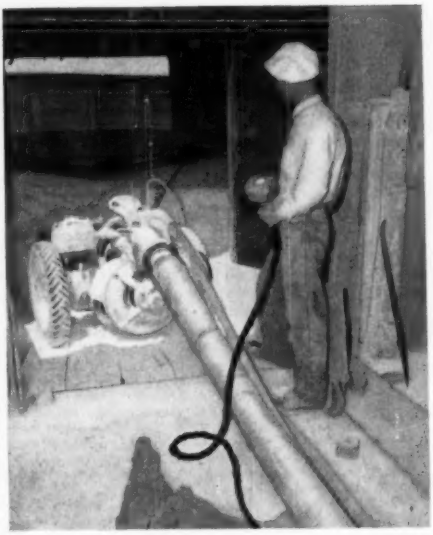

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Pricd 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size ¾ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that max..num air was entrained by the 28-48 mesh size sand rather than the 48-100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

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Use of Prepacked Aggregate Concrete in Major Dam Construction—Paul Baumann

A new method of designing flat slabs may prompt the revision of the present design methods specified in Sec. 709 of the ACI Building Code (ACI 318-47)

Rational Analysis and Design of Two-Way Concrete Slabs*

By C. P. SIESS† and N. M. NEWMARK‡

SYNOPSIS

A new method for the design of two-way building slabs is proposed and its step by step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

INTRODUCTION

This paper proposes a new method for the design of two-way building

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